सत्यमेव जयते

Ministry of Road Transport & Highways
Government of India

Capacity Development Project on Highways in Mountainous Regions

GUIDELINE FOR MOUNTAIN BRIDGE WITH ADVANCED TECHNOLOGY

Published by :
Indian Roads Congress

On behalf of the Government of India,
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Foreword

The road network plays a vital road in the overall development of a country by providing the access to employment, social, health and education. India has a vast area in hilly regions consisting of the Himalayan region from North to North-East, the Central Highlands of Aravali, Vindhya & Satpura ranges, the Sahyadri (Western ghats) and the Eastern ghats. Providing all weather connectivity road network in these hilly regions with extreme climatic conditions, difficult and hazardous terrain, topography and high altitude areas is a very tough task. The Ministry of Road Transport & Highways has always taken steps towards the enhancing the road network throughout the country and tackling the problem of roads in hilly regions of the country is one such step.

This Guideline for Mountain Bridge with Advanced Technology aims to improve the condition of bridge implementation on arterial national highway network and state highway network in hilly area in India by describing Japanese experiences and demonstrating Japanese latest technologies. It is prepared through the "Capacity Development Project on Highways in Mountainous Regions," bilaterally implemented by MoRTH and JICA. It also intends to provide the guidelines for control works, prevention works, for vast variety of terrains, severe climatic and river conditions, in hilly area of India, including Himalayas Mountains States, Western Ghats Mountain States, and North East States, such as Jammu-Kashmir, Himachal Pradesh, Uttarakhand, Maharashtra, Karnataka, Sikkim, Meghalaya, Tripura, Mizoram, Manipur, Nagaland and will be added as appropriate. It covers the aspects in various phases of implementation of the bridges in hilly area such as, planning (Project Preparation), design, and construction, as well as maintenance after opening.

The Japanese team as well as the Indian team, especially the Experts from JICA, Counterpart Officials from MoRTH, NHAI & NHIDCL, the Project Director and the Project Team, are to be complimented for their commendable effort in producing these guidelines. Their contribution would go a long way towards the sustainable development of roads in the hilly regions of the country.

The 27th day of January 2022

(I. K. Pandey)
Director General (Road Development) & Special Secretary
Ministry of Road Transport & Highways
Government of India

PREFACE

India is a vast country with diverse geological, climatic regions. Roads are one of the important elements of transport infrastructure for land surface movement of passengers, goods, ammunition & army –intercity, intracity, rural areas and neighboring counties. Indian road network passes through different terrains including hills and mountains. Mountainous regions consist of Himalayan region from North to North-East, the Central Highlands of Aravalli, Vindhya and Satpura ranges, Western and Eastern Ghats.

These mountainous regions, generally, have extreme climatic condition, difficult and hazardous terrain, topography and vast high-altitude areas. Roads in mountainous areas are often composed of multiple civil engineering structures including bridges and tunnels, and proper maintenance is essential to maintain the road function as a network. The Hill regions including the road networks are affected more frequently by floods due to torrential rainfall, landslides, snow avalanche etc. which compel many sections of road network closed during rainy season and winter season. However, accountability for providing safe, reliable and user comfortable road network ultimately rests with the Government departments.

For hill roads IRC has published numerous guidelines, manuals, codes, etc viz IRC: 52, IRC:48, IRC: SP:91; IRC: SP:108: IRC: SP:113, IRC: 5; etc for guidance and adoption by highway professional & road departments.

Ministry of Road Transport & Highways (MORTH), took up the initiative to frame Guidelines for Planning, Slope protection, Mountain bridge, Mountain tunnel & Operation and maintenance of highways in mountainous regions through technical cooperation from Japan International Cooperation Agency (JICA) in 2016. JICA took site visits of hilly roads in India, collected site problem statement, interacted with Indian experts from MORTH, NHAI, NHIDCL, PWD, IRC, etc and also conducted seminars, webinars, meetings, etc before finalization of following 5 documents;

- Guidelines for Planning
- Guidelines for Slope protection and embankment with advanced technology
- Guidelines for Mountain bridge with advanced Technology

- Guidelines for Tunnel on Arterial Roads in Hill Areas
- Guidelines for Operation and maintenance

The Guidelines for Highways in Mountainous Regions, as prepared by JICA mainly highlights appropriate planning, design principles for promoting safety conscious planning and design of roads, embankment, slope & its operation & maintenance aspect and will go a long way in guiding the highway engineers and planners to make our, hill roads, tunnels, bridges, safer, comfortable and of international standards.

It is quite natural, this JICA publications cannot be expected to cover all the subjects and details of highway engineering and as such should not be considered as a substitute for the Standards and Specifications as well as Codes of Practice.

The intent of this publications is to provide guidance to the engineers with the critical aspects of development of Highways in Mountainous Regions. These guidelines are dynamic in nature and the users, based on their experience, give appropriate feedback so as to enable the Guidelines to be revised/improved from time to time.

(S K Nirmal)
Director General (Road Development) and Special Secretary
Ministry of Road Transport and Highways &
Secretary General, IRC

Dated July, 2022

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List of Abbreviations and Acronyms

BOT	Build Operate Transfer
BRO	Border Roads Organization
C/P	Counterpart
DGBR	Director General of Border Roads
EIA	Environmental Impact Assessment
EPC	Engineering, Procurement and Construction contract
GNSS	Global Navigation Satellite System
GoI	Government of India
GQ	Golden Quadrilateral
HAM	Hybrid Annuity Model
HRB	Highway Research Board
IAHE	Indian Academy for Highway Engineer
IRC	Indian Roads Congress
JICA	Japan International Cooperation Agency
MoEFCC	Ministry of Environmental and Forest and Climate Change
MoRTH	Ministry of Road Transport and Highways
NHAI	National Highway Authority of India
NHDP	National Highways Development Project
NHIDCL	National Highways and Infrastructure Development Corporation Ltd.
PFI	Private Finance Initiative
PPP	Public Private Partnership
PSC	Pre-Stressed Concrete
PWD	Public Works Department
RCC	Reinforced Cement Concrete
VGF	Viability Gap Funding

CHAPTER 1 GENERAL

1.1 Background

1.1.1 India is on the way to implementing NHDP (National Highways Development Project) to strengthen the arterial road network for the nation and the majority of the routes planned have been implemented. The project started in 1998, and the implementation agency is NHAI (National Highway Authority of India) under the administrative control of the Ministry of Road Transport and Highways (MoRTH). It is expected to address the increasing traffic demand and sustain the economic development by strengthening the National highway network.

1.1.2 In addition to NHDP, Government of India incorporated National Highways and Infrastructure Development Corporation Ltd. (NHIDCL) in 2014, fully owned company of MoRTH to enhance rapid implementation of national highways and strategic roads in neighbouring area of international boundaries and also in North-Eastern Region.

1.1.3 Most of the remaining sections of NHDP, the majority of road implementation and improvement project of NHIDCL are in the hilly terrain and requires special countermeasure to cope with the natural conditions. Large scale of road structures, such as bridges, tunnels, high embankment/cut, etc. are required to satisfy the road geometry standards. Moreover, there are higher possibilities of natural disaster such as land slide, slope failure, earthquake, flash flood, debris flow, etc. Bridges in hilly area is required to counter those disasters up to certain level to satisfy the design standards.

1.1.4 Nearly 60% of NHDP has been completed and around 20% is in implementation (as on 31st May, 2017). The bridges in hilly area is implemented or under construction mainly in northern states of the nation, such as Jammu and Kashmir, Himachal Pradesh, and Uttarakhand, and also in the Western Gharts (Sahyadri) mountains region, such as Maharashtra and Karnataka. However, majority of the sections has not been completed nor started construction apart from the Golden Quadrilateral (GQ, Mumbai-Chennai section) and North-South corridor (Jammu and Kashmir). The situation in North-Eastern Region is approximately similar and the priority for implementation has given to the sections in plain terrain and not in the hills. It could be said that India is now completing the beginning phase of implementing bridges in hilly area and starting the development phase and will be followed by completion phase in not too distant future.

1.2 Scope

1.2.1 This Guideline covers the aspects in various phases of implementation of the bridges in hilly area such as, planning (Project Preparation), design, and construction, as well as reinforcement after opening.

1.2.2 The Guideline addresses the bridges in the mountainous terrain and steep terrain. (refer to IRC SP:73-2015 Manual of Specifications & Standards for Two Laning of Highways with Paved Shoulder for the details of mountainous terrain and steep terrain, hereinafter called IRC SP:73-2015)

Table 1-1: Terrain Classification

No.	Terrain Classification	Per cent cross slope of the country
1	Plain	0 to 10
2	Rolling	10 to 25
3	Mountainous	15 to 60
4	Steep	Greater than 60

(Source: IRC SP 73-2015)

1.2.3 The Guideline addresses the bridges which are the segments of the following road classifications, such as National and State Highways which is dealt in IRC SP:73-2015.

1.2.4 Referring clause 1.2.2 and 1.2.3, the design speed of the bridges dealt with the Guideline is 60 km/h. Other geometric features are also required to match the description of the same Manual (IRC SP:73-2015)

1.2.5 This Guideline covers bridges with the length of more than 6m referring to IRC SP:54-2018 'PROJECT PREPARATION MANUAL FOR BRIDGES' (hereinafter called, IRC SP:54-2018)

1.2.6 The contents for implementation of bridges have largely written in the relevant Indian Roads Congress (IRC) Codes as well as Highway Research Board (HRB), MoRTH publications as well as NHAI, DGBR Technical Instruction of BRO, IS Codes and other similar publications and technical papers available on the subject. The duplication of the similar contents is minimized and the reference is described as where indicated. This guideline intends to concentrate on the aspects in the hilly area. General and aspects in plain terrain will not be described in case of its necessity.

1.2.7 The Guideline consists of five chapters, such as General, Project Preparation, Design, Construction, and Reinforcement. The contents concerning Inspection and Repair of bride after the opening are described in "Guideline for Operation and Maintenance on Arterial Road in Hilly Area" that is prepared in this "Capacity Development Project on Highways in Mountainous Regions" (2018-2021, JICA) (hereinafter The JICA Project).

1.3 Objective

1.3.1 This guideline aims to improve the condition of bridge implementation on arterial national highway network and state highway network in hilly area by describing Japanese experiences and demonstrating Japanese latest technologies. It is prepared by JICA technical cooperation project in collaboration with Indian counterpart (C/P), MoRTH, NHAI, and NHIDCL in The JICA Project.

1.3.2 This guideline intends to provide the control works, prevention works, for vast variety of terrains, severe climatic and river conditions, in hilly area of the country, including Himalayas Mountains States, Western Gharts Mountain States, and North East States, such as Jammu-Kashmir, Himachal Pradesh, Uttarakhand, Maharashtra, Karnataka, Sikkim, Meghalaya, Tripura, Mizoram, Manipur, Nagaland and will be added as appropriate.

1.4 Types of Projects

1.4.1 This Guideline addresses the four following types of projects in implementing hilly roads.

1.4.2 Arterial Road Implementation Project, it represents the newly construction project of Arterial Road, National or State Highway, with totally new alignment and 2/4 lanes or more. It has a tendency to form massive projects with long section. The portion of the bridges physically and financially becomes minor. In this type of project rapid implementation will contribute to obtain social benefits. It also contributes to improve the financial balance if it is a toll road project. It is worth studying whether to apply the repetition of standard superstructure design; it would be a contributive method to shorten the construction period.

1.4.3 Arterial Road Widening Project, it represents the widening project of the existing Arterial Road, National or State Highway, without any/limited alignment change. The tendency of the project is relatively similar to clause 1.4.2 Arterial Road Implementation Project. However, applying the repetition of the superstructure design is not simple. It is worth considering making maximum use of the existing bridge if the other conditions permit, such as river flow, live load capacity, soundness of major members, etc.

1.4.4 Major Bridges Implementation, it represents the newly construction project of Major Bridge to cross wide valley to improve the exiting road alignment and shorten the road user travel time. The portion of the bridge tends to occupy the vast majority of the project. Selection of the bridge type considering its cost and construction time period requires to be studied very carefully to improve the economic and financial feasibility of the project. Challenging the new design and construction method also is worth applying since this kind of project requires tailor-made solution. There will be difficult constraints of maintenance after the opening of the bridge, therefore the effective and efficient maintenance method requires to be considered from design stage and necessary equipment is strongly necessary to be installed. The replacement in the future will be with extreme difficulty and it would be better to consider that there is very scarce chance to be realized. It is recommended that the service life (design life) of the bridge to be extended from the time span of the ordinary bridges. The extended time span requires to be defined in project preparation stage considering the importance of the bridge to the society, region and the nation.

1.4.5 Minor Bridges Implementation, Widening, and Replacement Project, it represents the small project which aims to improve the road alignment, flood flow condition, and traffic capacity of the exiting minor bridges. Therefore it tends to be relatively small project with limited technology requirement. It is likely that this type of projects are located in relatively small valley, the project type will be dependent on the existing relation among valley's river condition, road alignment, and the existing bridge width. If the road alignment requires considerable rearrangement or river cross section requires significant widening, the existing bridge needs to be replaced. If the exiting bridge's constraint is only the carriageway width, the existing bridge solely requires widening. In other case, if the river/creek/brook of the valley was solely treated by culvert not fulfilling the flood flow requirement, the new bridge shall be installed to widen the cross section and improvement of the existing road alignment ought to be studied consequently. The portion of the bridge tends to occupy the vast majority of the project. However, not like the Major Bridge Implementation, it is possible to define that project cost or social benefit produced by project is limited. Therefore, the factors to plan these kinds of projects are not only economic and technical aspects but also availability of the materials, contractors' skills, and needs of residents and society along the route.

1.4.6 Current stage of the Mountain Bridge implementation is starting the development phase as it is mentioned in clause 1.1. The Indian bridge construction industry is in the stage of accumulating its experience during the implementation of NHDP and other following projects. Current situation of the implementation of Mountain Bridge has several aspects to be improved, such as obtain alternatives of multiple types of bridges and eco-friendly construction methods. Indian construction industry has experience of constructing beam bridges both PSC/Steel, steel truss, PSC extra-dozed, ridged flamed PSC box girder, RCC/Steel arch etc. However the numbers are not so numerable and the beam bridges are selected in most of the cases. Moreover, the alignment is set in the terrain with not so steep terrain, therefore the construction constraints is not yet remarkably severe. To make forward progress and implement the highway to connect all over the trunk cities of India, accumulation of experience is still necessary to solve the difficulties of construction in extreme terrain conditions.

1.5 Definitions of Terms

1.5.1 Mountain Bridges; bridges located in mountainous and steep terrains. The Guideline addresses the bridges on the National and State Highways, and not the bridges on Major District Roads, Other districts Roads, and Village Roads.

1.5.2 Mountainous and Steep Terrains; terrains which has more than 25% of the cross slope of the ground.

1.5.3 National and State Highways; the highways defined as the National Highway by MoRTH, and highways defined as the State Highway by the Department of Transport of each State.

1.5.4 Mountain terrain; consists of mountainous and steep terrain.

1.5.5 Life Cycle Cost; cost for operation and maintenance of bridge for long period of time, such as 50-100 years. It includes, planning, designing, construction, maintenance, operation and rehabilitation cost

CHAPTER 2 PROJECT PREPARATION

2.1 General

2.1.1 Project Preparation signifies the phases of bridge project preparation from surveys to preliminary/basic design. Both technical and economic feasibility are required to be clarified. In case that the project is intended to be invested by private sector or public finance is reimbursed by the project's cash inflow, financial feasibility also shall be forecasted.

2.1.2 Project preparation for bridge project was guided in IRC SP:54-2018 'PROJECT PREPARATION MANUAL FOR BRIDGES' (hereinafter called, IRC SP:54-2018) for the general bridges including bridges in hilly area. This guideline concentrates on the mountain bridges and supplements the contents which are both required and preferred for preparing the projects of mountain bridges and also the projects which contains them.

2.1.3 The scope of project preparation includes the following works, data and information collection, survey and investigation, setting the design philosophy/policy and conditions, engineering/designing of road alignment, river/waterway training, section of bridges, slope protection, soft ground treatment, hydrological/drainage facilities, preparing drawings, bill of quantity, estimate implementation cost, and more. It also requires traffic demand forecast, environmental examination/assessment, land acquisition/resettlement plan, operation & maintenance works/schedule, social benefit and etc. for economic analysis. Toll rates, and interest rates, equity rates and etc. are necessary for financial analysis. The details of the work items in project preparation stage described above are described below.

2.1.4 The objective of the project preparation is to define the scale of the project and secure the technical/economic, and financial if necessary, feasibility for the project to secure the implemented bridge with sufficient soundness and safety to the users, to produce the social prosperity/benefit throughout its lifespan. It also aims to prevent the project from terminating before opening, the project/investor from bankrupt/cash shortage during the operation, or left behind without sufficient numbers of the users and producing scarce social benefit.

2.1.5 The objectives mentioned above are studied and calculated and described in Pre-feasibility Report and Feasibility Study Report. The detailed procedure of preparation of both reports is explained below.

2.1.6 India is starting the stage of implementing hill expressways and arterial highways with fast design speed, such as from 60km/h to 100km/h or even more in some circumstances. Therefore, it can be said that the whole country is now learning how to prepare the project of mountain bridges and projects which include mountain bridges. Project preparation can be done by Indian consultants/engineers alone, however collaborating or accepting advices from experienced foreign consultants/engineers from mountainous developed countries could be considerably effective and efficient for not only the project preparation but also the entire project, since the results of preparation could affects remarkably until the operation stage both favourably and unfavourably.

2.1.7 Apart from receiving assistance from foreign expatriates, securing sufficient period and budget is very essential in preparing project related to mountain bridges. Each bridge in mountain terrain has unique natural and social condition which requires carefully consideration. In case of the preparation are made based on the projects in plain terrain, even mountain terrain of different region, it could miss out very important conditions for the project and could cause serious complication in project implementation/operation stages.

2.1.8 Mountain bridges tends to cost more than bridges in plain terrain both in rural or urban area due to its severe natural and social environment. For instance, every material costs more due to its long distance of transportation to the site from the manufacturer. Project preparation reports are evaluated by evaluators from different background and level of engineering knowledge from different authorities. Therefore, unique characteristics and needs of the project require to be presented in project preparation report in simple and easily comprehensive manners so as to obtain projects approval with sufficient budget.

2.1.9 The stage of project preparation is following three stages.

i) Pre-feasibility study, preparing pre-feasibility report
ii) Feasibility study, preparing feasibility report/preliminary project report, and
iii) Detailed preparation, preparing detailed project report
(referring to IRC SP:54-2018)

Chapter 2 explains the pre-feasibility study and Feasibility study which are the step to identify the contents of project and justify the implementation. Detailed preparation which prepares the Detailed Project Report includes detailed designing. Therefore, it is described in the following Chapter 3.

2.1.10 Pre-feasibility stage is to identify the project. In case that the project is to implement long-span bridge or numerous span bridge, the ratio of bridge in the project will be majority both in the cost and work quantity. The possibility of natural disaster, such as land slide, debris flow, snow avalanche and etc. requires to be avoided when planning the bridge locations and crossing locations. These kinds of natural disaster tend to damage the bridge very severely and the road will be closed for long period which would affect regional economy negatively. The reconnaissance of the site is much more important than that in plain area. Details of the pre-feasibility study are described in Subchapter 2.2.

2.1.11 Feasibility stage is to conclude the project technical and economic/financial feasibility from the collected data/information, design, and analysis/study. Therefore the preliminary survey is required to study the technical feasibility and forecast the economic/financial feasibility. However, since it is still in very early stage of the project, excessive amount of survey is not necessary. Yet it is expected to execute more careful survey of natural condition than that of bridges in plain terrain due to its complexity. Duplication of other projects' study results often does not function due to its unique terrain and natural condition of the site. Details of the feasibility study are described in Subchapter 2.3.

2.1.12 Detailed preparation stage, which is described as 'detailed engineering stage' in the IRC SP:54-2018, is to prepare the information for implementation of the project. Therefore, more detailed survey, design, and analysis/study shall be carried out compared to that of feasibility study. Proper attention shall be paid for project implementation cost since insufficient estimate could end up in shortage of budget during the construction. The mountainous terrain could be very different in geological condition by locations and rough survey, design, and analysis/study are likely to be a cause of enormous additional budget for completing the project. Even the construction were completed, lack of budget could lead to low quality or durability of the bridge and its surrounding facilities. This issue would increase life cycle cost of the project and give negative impact to the regional economy. This stage in the mountainous terrain needs closer attention than that in the plain terrain. Details of the detailed preparation are described in Chapter 3 due to recent change of the contact scheme in highway/expressway implementation which combines detailed design works in construction contact, such as PPP, EPC, and HAM.

Table 2-1: Stages in the Project Preparation in Mountainous Terrain

No.	Stage	Purpose
1	Pre-feasibility Study	The main purpose of this study is project identification by executing setting the candidate sites of bridge avoiding natural disasters (special attention is required to the disasters in mountainous terrain, such as land slide, debris flow, etc.), and reconnaissance of the site
2	Feasibility Study	This study's main purpose is securing technical and economic/financial feasibility by preliminary site survey, design and analysis. More cautious attention is demanded for mountain bridges than the bridges in plain terrain. Duplication of other projects' study results often does not function effectively.
3	Detailed Preparation	This study is the last stage of project preparation and prepares necessary items/documents for actual project implementation by executing site survey, design, analysis. Study with the results from rough site survey shall be avoided since it would leave serious issues for project implementation/operation and also deaccelerates economic growth of the region. This stage in the mountainous terrain needs closer attention than that in the plain terrain.

note) Detailed preparation is described in Chapter 3.

2.2 Pre-feasibility Study

2.2.1 As it is mentioned above and in the IRC SP:54-2018, Pre-feasibility Study is done for Project Identification. Pre-feasibility study is the first step of the project. The study report contains basic and initial information of the project. The selection of the crossing location of the bridge and project site needs to be done based on the technical and social aspects, such as natural condition (river flow, bank stability, geotechnical condition, rainfall intensity, etc.), natural disaster history, traffic demand, road category, land acquisition, involuntary resettlement of inhabitants, contribution to regional development, etc.

2.2.2 Indian mountains, represented by Himalayan Mountains, are very new mountains with very steep slope. It tends to slide and collapse with saturated underground water. India has heavy rain season every summer due to the presence of the monsoon from Indian Ocean towards Himalayan Mountains. The slopes tend to be in condition that can end up in disaster. Moreover, rivers in steep valley have very high discharge with high velocity in rain season. The flood depth can rise significantly and rapidly. It also erodes the river banks very rapidly and widely. These facts need to be considered carefully in selecting the crossing location of mountain bridges to avoid severe damage in implementation as well as in operation.

2.2.3 Due to the facts mentioned in 2.2.2, it is very difficult to indicate the most appropriate crossing location for the mountain bridges in the pre-feasibility study. Therefore, several alternatives, such as 2 to 4 different locations, shall be proposed so as to study and select the most appropriate crossing location in feasibility study. These 2 to 4 crossing location alternatives shall be selected in the pre-feasibility study among possible crossing locations for the project. The factors to be considered in identifying the alternative alignment are shown in Subchapter 2.2, Pocket Book for Bridge Engineers, 2000 (hereinafter called POCKET BOOK-2000).

2.2.4 This selection of candidate of crossing location could be done by evolving the matrix with the factors shown in 2.2.1 as it is described in the IRC SP:54-2018. The selection could also be made with two-step selection when the numbers of the possible crossing location is large. Two-step selection is done with two different selection policy, the first by document/data study and the second by site reconnaissance for instance.

2.2.5 There are significant needs to cross wide rivers with high discharge since these rivers separate the region almost completely. However, the span of the bridge crossing these rivers would be very long and the project cost and the ratio of bridge in the project would increase considerably. The selection of the crossing location alternatives shall be done with careful attention in preparing the project of these cases.

2.2.6 There will be mountain bridges which do not cross obvious river in preparing high category highways with high design speed. These bridges are for improving the road alignment and basically set along the slope. It could end up with the bridge with large numbers of spans. The location and the length of these bridges largely depend on the road alignment. Therefore, the alternatives of the road alignment shall be proposed instead of the crossing section alternatives. The road alignment shall be cautiously studied and selected and the bridge section is set accordingly in the feasibility study.

2.3 Feasibility Study

2.3.1 General

2.3.1.1 As it is mentioned in the IRC SP:54-2018, feasibility study in the project preparation study is named 'Techno-economic Feasibility Study". It is aimed to indicate the technical as well as economic feasibility of the project. Unlike in the era when the bridge engineering was founded, the most of bridge projects are technically feasible in the present days by spending unlimited budget. However, the budget needs its ceiling to formulate realistic implementation plan of road network. New implementation of bridges in hilly area could cost much more than the budget of the bridges in plain area, especially in the Himalayan Mountains. The crossing location and bridge site requires close investigation to be determined to secure the economic feasibility.

2.3.1.2 The highway network followed by the expressway network for the entire nation is profoundly significant and indispensable for regional development without polarization. Roads with high design speed are not able to be designed with curves with small radius. Therefore it requires bridges and tunnels to connect two regions separated by mountain range. The given conditions for bridge implementation in hilly area are considerably different with the bridges in plain terrain in the aspect of natural and traffic condition. Natural condition tends to be severer with less traffic volume and demand. Considering these conditions there are more limitations for bridge projects in hilly area to be economically feasible satisfying design and construction requirements excessively. Saving implementation cost is much more required than the bridge projects in plain area.

2.3.1.3 As is mentioned in above 2 paragraphs, the trade-off between the technical and economic aspects is very important issue to be solved in the preparation stage, and needs careful investigation and analysis specifically in the feasibility study as it is the major evidence to give green light for project implementation. The reliability of the justification of the project implementation highly depends on the accuracy of the feasibility study. Bridge projects in hilly area are more dependent on the result of the feasibility study than that of plain terrain. The engineers as well as project owners shall recognize this fact and execute the study not taking the implementation as a granted since the contribution to regional development is one of the most important social demands.

2.3.1.4 This Subchapter's structure follows the structure of the Chapter 5, IRC SP:54-2018.

2.3.2 Identification of Data Needs

2.3.2.1 As is mentioned in IRC SP SP:54-2000, the data collection is very important work in the study to make the result reliable. Therefore, it is necessary to understand which data is required in which level. Identifying the data needs in the beginning of the study is essential. Bridges in hilly area tends to have its unique conditions by site in aspects of nature, procurement, and labour. Therefore, understanding of the potential crossing locations are required to improve the effectiveness of data collection.

2.3.2.2 However, it is impractical to assume that the engineers performing this study can understand the site characteristics only by studying the data in the past. Therefore, the modification of the identification of data needs and the plan of data collection is necessary after the commencement of data collection, site investigation, engineering, and evaluation when required. Although this is very basic for all the bridge projects, special care is required for bridges in hilly area since natural condition is much severer and the engineers are often less unfamiliar to the region of the projects than in plain area.

2.3.2.3 The effectiveness of the data collection is also an essential factor to be considered in planning the data collection. The existing data shall be made good use of in this study. Collecting the useful data and information from relevant authorities could reduce the volume of site investigation which also contributes to cost savings of the study.

2.3.2.4 Since the potential crossing locations in the mountains are not often well surveyed locations, it could be estimated that there is very limited data for demonstrating the site conditions. The data/information collection could highly depend on the site investigation for obtaining reliable study result. However, executing site survey in hilly area, especially in steep mountains, requires higher cost and longer period of time than those in plain area due to its severe natural condition. Trade-off between improving the quality of the data/information for the potential sites and saving cost and time for the project preparation is essential issue to be solved namely at this stage of the study, and continuously improved during the study.

2.3.2.5 Once bridges are implemented, it lasts for decades due to its long life span. Since the sustainable usage and viability are taken as granted for road bridges, the engineers as well as the projects owners tends to collects namely the data for construction and pay less attention to the data for operation stage. It is requested to collect data for operation and maintenance stage, specifically for the region which includes potential crossing locations, for maintaining the service level for the entire life span of the bridge. Bridges in hilly area are likely to face more severe natural conditions against deterioration and collapse than the bridges in plain area. The causes of the severity vary by region but namely are intense precipitation, flood with debris, avalanche, slope failure, and earthquakes.

2.3.3 Data Collection

2.3.3.1 Maps, Plans and Topographical Features

(1) Index Map

Index map is prepared to indicate the location of the bridge and the entire project as well as the access route from the capital and major cities near by the planned location of bridges implementation. Additional to the items which are described in IRC SP:54-2018, Subchapter 5.3., the shape of ranges and valleys related to the planned bridge location in terms of river/hydrology, geology/geotechnics, and vegetation/land-use are recommended to be shown in the map for the bridges in hilly area. Since the wide area shall be shown in this map, the scale of the map shall be studied carefully not only by the scale of the planned bridge, but also by the geographical characteristics of the region and chosen from 1:50,000 to 1:250,000.

(2) Site Plan

Site plan is prepared to show the details of the crossing conditions, such as river, valley, slope, road, railway, and etc. However, most important crossing condition of the planned bridges in hilly area is river. Therefore, minimum range of the river which needs to be shown in the plan is 100m for both sides of the steam (upstream and downstream). However, the range shall be extended up to 500m for the bridges which crosses major valleys/rivers. The width of the valley/river shall be paid more attention than total length of bridge as written in IRC SP:54-2018. This is not the case if the river is well-trained for considerable section including the planned bridge location. However, in the deep mountain region in India, this case rarely exists currently and the description of wide range of valley/river is very important in preparing site plan.

(3) Contour Survey Plans

Contour plan is to show the details crossing stream with its topographical features. The distance to cover by this plan is clearly mentioned in IRC SP:54-2018. However, the topographic features of the site may vary significantly by location in hilly area. The coverage area of the contour plan shall be considerably studied not to fail in reporting the influential features. The range of distance shown in the contour plan is defined from 100m to 1.5km by the size of catchment area in IRC SP:54-2018. However, large discharge could occur in hilly area since the gradient of the stream is very steep. The engineer responsible for the design of the river cross section (after trained, if necessary) and the clearance of bridge shall study the necessary distance to be shown on the contour plan.

(4) Topographical Map

Topographical Map is the base of every map prepared in the study. Therefore, accurate and updated data shall be referred to the map. Ground survey is most accurate and detailed and it shall be recommended to be done from the very essential stage of the bridge implementation. However, the terrain and geographical condition of the site may not allow executing the ground survey for the bridge projects in hilly area in details due to budget constraints. Therefore, aerial photos or satellite images could be taken as an alternative to prepare the map. However, imaginal survey does not reproduce the ground level covered by thick vegetation. Therefore, the engineer responsible shall consider the difficulties of imaginal survey and design the bridge in fertile vegetation area. Moreover, the image which is selected for the map shall not be out-of-date since the ground in hilly area, especially the slope, are very fragile in Indian mountainous regions due to tectonic deformation and heavy rainfall of monsoon.

(5) Catchment Area Map

Catchment area map is a very important map to design discharge and to understand the characteristics of the flow. This map is required for the hydrological/hydraulic study of the bridge. Since the condition of the catchment area, such as terrain, rainfall, gradient, vegetation, and etc., differs significantly in hilly area with plain terrain, more detailed information shall be demonstrated in this map than the map for the bridge in plain terrain. However, if the crossing item is not a river, this map is not necessary even in the hilly area.

(6) Geological map

The hilly regions in India are fragile to many kinds of natural disaster, such as flash flood, debacle, land slide, slope failure, debris flow, etc. These kinds of disaster are caused by rainfall and its geological condition at the site and its upstream area. Geological map for bridges in mountainous area is essential to understand the natural condition of the site. Therefore, the engineers responsible for the project shall collect the map with sufficient information both in quantity and quality.

2.3.3.2 Traffic Data

(1) Traffic data for the bridges in hilly area are required to design the typical cross section of the bridge. It is required to have the capacity for the traffic demand in future, the target year. Therefore, the traffic forecast demand shall be calculated based on the traffic data collected.

(2) If the bridge forms a part of the highway project, traffic data, traffic demand forecast results, as well as typical cross section of the highway project can be utilized directly, as is mentioned in IRC SP:54-2018.

(3) If the bridge is planned specifically without the surrounding highway implementation/improvement, traffic data shall be collected and the traffic demand forecast study shall be executed to provide the justification to the design of the typical cross section.

(4) If the data which could be collected is not sufficient or updated enough to conduct the traffic survey, though it is most probable in the rural regions, it is requested to conduct traffic count survey to improve the quality and reliability of the traffic demand forecast.

(5) Socio-economic statistics are required to improve the quality and reliability of the traffic demand forecast, particularly if the reliable traffic data is only the result of traffic count survey conducted for the project. The data of single point of time is not quite adequate for calculating traffic demand forecast without any complementary data, such as socio-economic statistics.

(6) It is also important to consider the seasonal/daily fluctuation of traffic survey results if the data collected or the traffic count survey does not reflects these fluctuations, though it is most probable in the rural regions. It is accepted to adjust the data by using socio-economic statistics to speed up the implementation, since it requires years to collect sufficient data for seasonal/daily fluctuation.

(7) It is requested to consider assigning an experienced traffic engineer for the traffic demand forecast study to compliment the luck of data required in hilly area, not like the main corridor of the national highway network.

(8) The data required are followed as is mentioned in IRC SP:54-2018,

Table 2-2: Required Data for Traffic Demand Forecast

a) Classified Traffic Volume Count
b) O-D Survey
c) Speed and Delay Survey
d) Pedestrian Volume Survey
e) Socio-economic profile of the project influence area for the last 5 years

2.3.3.3 River Cross-section for a Bridge Project

(1) Defining river cross section for the bridge project in hilly area of India requires consideration of seasonal fluctuation caused by monsoon. Therefore the data collected shall be of the rainy season with large discharge.

(2) River Cross-section of the bridges in hilly area need to be defined and planned as linear waterway at HWL without any restriction, as is mentioned in Clau 106.5.1, IRC:5-2015 'Standard Specifications and Code of Practice for Road Bridges, Section I, General Features of Design' (hereinafter called IRC:5-2015).

(3) To define the river cross-section of the bridges in hilly area, the existence of river training plan shall be studied. If there is a river training plan, river cross-section of the bridge shall follow the cross-section defined in the river training plan. If the river training has been executed, river cross-section refers to site condition.

(4) If there is not any river training plan nor river training has been done, the river cross-section of the bridges shall be defined based on the discharge of the river and characteristics of the river flow. For the data collection of the discharge of the river and characteristics of the river flow, refer to the next clause, 2.3.3.4.

(5) If the bridge is planned in very hilly terrain with steep slope, it tends to have various river cross-sections both in upstream and downstream of the candidate crossing section of the bridge. Therefore, to define the linear cross-section of the river adjacent to bridge crossing location, varied flow calculation is recommended to define HWL, velocity of flood flow, gradient of trained river, and river cross-section. At least 1 cross section each both from upstream and downstream is required for executing this analysis.

2.3.3.4 Hydrologic Data and River Characteristics

(1) Hydrologic data for the bridges in hilly area, specifically precipitation data, shall be collected from the observatory located upstream. If there is no observatory locate upstream, data from nearby observatory in same catchment area could be used. When there is not any observatory located in the same catchment area, the data from the observatory in adjacent catchment area will be the only reliable data for planning the bridge river conditions. However, the data shall be adjusted to suit the bridge site by well-experienced engineer, i.e. senior hydraulic engineer.

(2) Historical record is the main data for the planning the river cross-section of the bridge. However, the data seems to be getting severer due to global warming and climate change. Especially the rainfall intensity is likely to increase in many regions in the world, and heavy torrential rainfall damages the neighbouring area of rivers including bridges. Bridges in hilly area are prone to get damage not only by flood flow but also by sediment/mud flow. Therefore, adjustment of the precipitation data and discharge analysis should be considered for bridges for important/arterial roads.

2.3.3.5 Catchment Area Characteristics

(1) Physical characteristics of the catchment area impact the river flow/run-off, especially the flood flow. Catchment are in hilly area tends to have very steep slope with natural land use. Gradient of slope shorten the flood concentration time and makes larger run-off with smaller rainfall.

(2) Vegetation of catchment area is another factor to characterize the catchment area. Fertile vegetation absorbs the rainfall and reduces the discharge and barren land/desert does not reduce the discharge as much. Mountainous regions in India have very different vegetation characteristics by region of which the characteristics in flood need to be studied carefully.

(3) Soil condition of the catchment area will also affect significantly in flood flow. Debris, sediment/mud flow is likely to occur in fragile soil. Hilly areas in the Himalayan Mountains has very strong tendency of having fragile soils due to its land deformation. Constructing substructures shall be avoided in the area with large size of debris and large scale of sediment/mudflow.

(4) Some catchment area is developed for agricultural use, such as cultivate land or urbanized area and will increase the run-off and shorten the flood concentration time significantly. Developed catchment areas in hilly area are usually with slopes and have short flood concentration time with large discharge of its size. Land use investigation is one of the most important data collection works for bridges in hilly area.

2.3.3.6 River/Channel Characteristics

(1) River characteristics in hilly area are strongly affected by hydrologic data, catchment area characteristics, and soil condition along the river. To characterize the river, these sets of information shall be carefully collected.

(2) River characteristics impact not only the river cross section but also the location of substructure. If the placement of substructure is restricted, the span length of the bridge will be increased. This will define the superstructure type and considerable affects the construction cost. Bridges in hilly area with steep slopes and fragile soil are prone to have longer span due to its river characteristics. Hilly areas in the Himalayan Mountains are the typical example of this issue.

2.3.3.7 Flood Flow Data

(1) Collecting the flood flow data is very essential in defining the flood flow/discharge and other hydrologic condition for bridge planning/designing. If the flood is recorded in adjacent observatory, it is the most reliable data to be based on. If the recorded data is not available, historical data, as is mentioned in Clause 5.3.7, IRC SP:54-2018, could be very useful information to be utilized.

(2) Historical data can be collected at the river facility adjacent to the planned river crossing location of the bridge. Historical marks at the site or local enquiry are alternative methods for collecting flood data, but the accuracy is very limited. If the road category of the bridge shows the importance of the route, they shall not be the only evidence to relay on. However, the data shall be adjusted to suit the bridge site by well-experienced engineer, i.e. senior hydraulic engineer.

(3) Flood Discharge Data of nearby gauging points are commonly used in flat and rolling terrains. However, it is not recommended to rely on the data of other points of the river in mountainous and steep terrain because it could differ significantly by points.

(4) When reliable recorded data or historical data is not available, theoretical analysis with rational method using related formula would be the most reliable methodology in planning bridges. However, site investigation shall be done to justify the theoretical analysis.

2.3.3.8 Sub-soil Data (extended to soil/geological data)

(1) Soil/geological data are very important for planning bridges in hilly area. The data is not only the base of bridge's stability but also of span arrangement (including approach alignment), construction planning, material purchase, river training,

(2) Sub-soil data is basically collected by geological survey based on the boring survey and lab test executed in laboratory. This is very indispensable information for bridge planning/designing/construction and cannot be extracted. However, there are some cases the existing data are available, such as the data from the boring survey which was executed for the existing structures, bridges, dams, revetment, and even buildings nearby. These data is very useful information to decide the river crossing location and also further boring survey location at the candidate sites.

(3) Also geological profile of the candidate site or vicinity area is useful for understanding the structure of sub-soil for the area including candidate site. It is prepared in major tunnel, dam construction projects when large number of boring survey executed.

(4) Land slide/debris flow disaster risk area is one of the most important data to collect in selecting bridge site. These risk areas shall be avoided for river crossing locations to save the bridge from the disaster. Topographic map is useful to filter the risk area for risk mitigation. If the area cannot be avoided, the data to plan/design slope protection shall be collected in site survey.

(5) Scouring depends heavily by the soil at the site. Types, characteristics, particle size distribution, etc. shall be collected. Since the foundation of the bridges in hilly area tends to be shallow, and the flood velocity tends to be rapid, foundations of the bridge in hilly area are prone to face the rick of scouring. Therefore, it is very important to collect and understand the behaviour of the soil in flood from the surface to base of foundation. It shall be analysed by the data of the detailed investigation at site. However, if the boring survey has been executed previously, it can be very helpful information for further stage of the study.

2.3.3.9 Meteorological Data

(1) Meteorological Data of same catchment area is the most favourable data to be collected in the stage. However, it can be the data of the vicinity when the distance of the observatory is long or the data do not exist.

(2) Precipitation, temperature, wind, humidity, sunshine duration, etc. shall be collected for hydrological analysis of the crossing location, structural calculation, construction planning, surface drainage, etc. The meteorological condition tends to be different by location in precipitous mountain areas with high altitude. The data collected shall be justified after the site survey to use in the design in further stage. If it is far from the site condition, adjustment shall be done by experienced meteorologist.

(3) Wind tends to flow 3-dimensionally in deep valleys with steep slopes. Therefore, collected wind data shall be justified after the site survey or adjusted for setting the conditions in design and construction planning. This could be critical in very high bridges.

2.3.3.10 Construction Materials

(1) Construction Materials for bridge are mainly cement concreate and steel. Therefore, availability of aggregate and cement shall be collected, aggregate could be procured from the borrow pit adjacent to candidate site usually for the bridges in hilly area. Cement is fabricated in the factory usually near the lime stone mine. The experiences of the past construction projects are very helpful and the data shall be collected from relevant authority/organization. It requires large amount in short period in constructing major bridges, therefore not only the location of cement fabricator but also the production capacity shall also be collected.

(2) Location of steel member manufacturer, such as plate girder, rebar, that has large quantity is data which shall be collected. Most of the superstructure of the bridges in hilly area is made of cement concrete and required steel reinforcement members. Also there are some locations steel superstructure exceeds the concrete superstructure both in technical and economical point of view. However, cost of transporting steel for major distance tends to be very high. Therefore, location and capacity of the manufacturer shall be collected. The experiences of the post construction projects are also very useful.

2.3.3.11 Environment Data

(1) Not only bridge projects but also the highway projects in hilly area are prone to leave negative impacts to the natural environment of the site. Therefore, when the project is bigger than specific level, 'Environment Impact Assessment' (hereinafter EIA) is necessary. Bridges in hilly area tends to impact more to nature than the bridge in urban area. The information whether obtaining the approved EIA is necessary or not is very important information to be collected. IRC:104-1988 'Guidelines for Environmental Impact Assessment of Highway Projects' (hereinafter called IRC:104-1988) and other relevant regulation shall be collected. Ministry of Environment, Forest and Climate Change has the responsibility of preserving the environment and shall have strict regulations.

(2) If the approved EIA is necessary, the data to plan the EIA preparation study is necessary. The details shall refer to IRC:104-1988.

(3) If the EIA is not necessary, the existence of rare and endangered species both flora and fauna in and around the site shall be checked by documents. When the site is in or near the habitat, detailed study shall be planned following relevant regulation to protect these species.

(4) The land acquisition is always necessary in bridge implementation no matter where the site is. Therefore, land use data shall be collected to study the social environmental impact typified by local inhabitants' resettlement. When the numbers of the effected inhabitants exceed certain number, resettlement action plan shall be studied following relevant regulation (which ministry?) in further stage. Hilly region tends to have small area of inhabitable land and could be very difficult to prepare the land for resettlement nearby, therefore, these data shall be collected in as early stage as possible to review the road alignment before selecting the site of bridge.

2.3.3.12 Special Design Requirement

(1) Data to judge necessity of special design requirement shall be collected. Most of the cause for necessity is severe natural condition, such as heavy rain, strong wind or earthquake. The experiences of the post construction projects are the most reliable data. However, there are disasters of which the scale/damage exceeds the historical most critical event in recent years, and it should be taken in account that only the past experience data would not be sufficient in defining the critical design conditions.

(2) Moreover, the data to plan how to study the special design requirement shall be collected when it is judged necessary. Most reliable method is execute model study as is mentioned in Clause 5.3.12, IRC SP:54-2018, with requirement of careful application of the result and large cost.

2.3.3.13 Existing Services

(1) Location of life line facilities shall be collected from relevant authority/organization, such as electric lines, water lines, optic information lines, etc.

It will be the base of the relocation plan and works before and during the construction works. High tension lines from hydrological power plant are most likely to be the most important line for the bridge project in hilly area.

(2) Location of buried life line shall also be collected for location of substructure, construction regulation, etc. Bridges in hilly area sometimes are adjacent to residential village and have buried life lines. It is not as serious as bridges in urban area, but it is the information which restricts bridge implementation and shall not be ignored.

(3) Since India is economically developing rapidly, there shall be future implementation plan for life line facilities, including mega-size project. The existence and outline of these plans shall be collected so as not to make conflict in implementing both projects in future.

2.3.3.14 Labour Accommodation

(1) Labour for bridges in hilly area tends to be very scarce due to small population density in the site region. It is more obvious for the bridges in expressways/arterial highway because it usually runs far from residential area to mitigate negative impact, such as noise, vibration, air pollution, etc. Therefore, it usually required labour accommodation nearby the site. The availability of location and materials needs to be collected.

(2) Not only the amount of labour for the project, the quality of the labour needs to be collected. In the hilly area it is not rare that purchasing skilled labour is very difficult within the region. Many of them shall come from urban area with distance. Experiences of the contractor for the previous projects are the most reliable data to be collected.

(3) Workers usually stay for long time at the site in big projects. Therefore, the level and availability of living standard are requested to be collected. Sanitary and medical environment shall be kept in high level equivalent to civilized urban area to maintain the workers' health, and also quality of non-working time shall be taken care for workers mental health.

2.3.3.15 Other Authorities

(1) There are many authorities which will approve the activities in bridge/highway project apart from the authorities of transport sector. Even as governmental authorities, there is central, provincial, and local authority. They have very important information and data regarding the data collection mentioned in previous clauses. Information of 'who has what' will be very powerful in implementing the project and shall be collected.

(2) It requires many approvals from other authorities to implement bridges, and relevant organization shall prepare the allocation documents. The detailed information will ascertain their specific requirements. The attention shall be paid that the requirements in hilly area are not the same with that of urban or plain area.

(3) Regions in hilly area are usually very far from the capital, Delhi. Communication among central and rural authorities tends to lack in quantity and quality. Interaction and collaboration with these related authorities shall be constantly executed.

2.3.3.16 Data regarding the Existing Bridges

(1) Every data regarding the existing bridge in the vicinity is useful in new bridge implementation. Reports and documents from planning to maintenance stage shall be collected to understand site condition. Site condition in hilly area is usually severer than the knowledge of the engineers who collect the data. Interviews and inquiries to the engineers who worked for the existing bridge project are also helpful in collecting data.

2.3.3.17 Construction Conditions (added)

(1) Sites in hilly area have strict spatial limitation. They tend to require working space on the slope with large height from the bottom of the valley. However, construction sites require certain area of flat land as construction yards to allocate construction machinery. Therefore detailed topographic map with contour, the map which shows the natural reserve area shall be collected.

(2) If the bridge/highway is completely new, the access route to the site also needs to be constructed prior to main construction. Detailed topographic map with contour which indicated natural reserve area shall be collected for further stage of the study.

(3) Countermeasures to mitigate the negative impact to natural environment restrict considerably the construction works. Data of rare and endangered species, special requirement from provincial/local government, and experience of the contractors in previous construction project are very useful and shall be collected.

(4) Construction condition can be a one of the prominent factor to plan the bridge in design and construction. Bridges in steel hilly area has wide variety and strict limitation in construction. Therefore, the bridge engineer shall keep this in mind from the very beginning stage of the project and proceed the study/planning.

2.3.4 Organization of Activities

2.3.4.1 Major activities required for preparing the feasibility study report are identified in IRC SP:54-2018 as follows. This guideline will add and modify slightly to apply for the bridges in hilly area. *Italic letters underlined* are the amendments.

Table 2-3: Required Activities for Preparing the Feasibility Study Report

a)	Collection of data
b)	Preliminary *site* investigations
	Topographic survey
	Hydrological survey
	Traffic survey
	Sub-soil *(Geological)* investigation *based on boring survey*
	Survey for land acquisition and resettlement
	Environmental survey
c)	Analysis of data
	Site reconnaissance
d)	Identification of alternative bridge sites/alignments
e)	Development of design philosophy
f)	Preliminary design, general arrangement drawings and preliminary cost estimates
	Preliminary comparison study for bridge structure
g)	Environmental impact examination and economic evaluation and financial evaluation (for PPP)
h)	Model study where required through appropriate agency and taken up as a parallel activity to items (e) to (g).

a) Details of data collection are mentioned in previous clause 2.3.3.

b) All of the preliminary investigations shall be done at the site, not remotely by documents/internet. It has to be done to fulfil the needs to execute the preliminary design. The details are mentioned in the following clause 2.3.6

c) Site reconnaissance shall be done by responsible engineers/specialists of each field, such as bridge, highway, hydrological, geological, environmental, resettlement, procurement, and economist to confirm the data collected is sufficient and also to justify the quality. If it cannot be achieved, additional site survey shall be conducted before the commencement of preliminary design.

d) Identification of alternative bridge site/alignment is done in this step followed by the selection of the bridge site and basic alignment including approach. If the bridge forms a section of arterial highway/expressway, alignment of the highway/expressway is given condition for bridge planning, and adjustment along this alignment is done in bridge design stage. There are no needs to identify bridge alignment without considering contiguous highway/expressway section.

d) If the bridge is major bridge with very long span i.e. more than 200m, and the length has very scarce opportunity to reduce largely, the location and alignment of the bridge shall be dominant to set highway/expressway alignment. Identifying the alternative bridge site/alignment is required prior to setting highway/expressway alignment in this case.

d) Recommendation of selection procedure is as follows. At least 2 to 4 alternative sites shall be set and each site shall have plural alignment alternatives. Selecting the most appropriate site/alignment shall be followed before preliminary design. Comparison factor shall be structure stability, constructability, accessibility, procurement easiness, cost, maintenance easiness, economic efficiency (EIRR), environment, aesthetic, and more specific factor related to the region. Results of comparison study shall be proposed after selecting the most appropriate site/alignment using quantification method and decision matrix systematically with engineering point of view.

e) Details are described in following clause 2.3.7

f) Details are described in following clause 2.3.7

g) Details of environmental impact examination are described in following clause 2.3.7, and of economic analysis are in clause 2.3.8.

h) Necessity of the model study shall be done by the results of data analysis. If there are fundamental issues which cannot be solved by conventional preliminary investigation and design, executing the model study shall be considered with budget constraint.

2.3.5 Source of Data

2.3.5.1 Example of the source of data is described in IRC SP:54-2018 as follows. This guideline will add and modify slightly to apply for the bridges in hilly area and make it more in details. *Italic letters underlined* are the amendments.

Table 2-4: Example of the Source of Data for Feasibility Study

i) Maps *(Topographic with contour, Geological showing Land slide/debris flow disaster risk area, Vegetation showing habitats of rare and endangered species, Meteorological showing the rainfall intensity, etc.)*
ii) Records *(Meteorological, Disaster including flood, earthquake, cyclone, etc.)*
iii) *Data provided by* Govt. Depts. such as Industries, Agriculture, Planning Board, *Environment, Forest, Land resources, Energy, Railways, Earth science,* etc.
iv) Field inspection *(details described in previous clause 2.3.4)*
v) Information obtained from local enquiry *(specifically disaster related information, such as flood, earthquake, avalanche, land slide/debris flow, etc.)*
vi) Geological details *(Boring survey, Laboratory tests, Academic research and stocks, etc.)*
vii) *Satellite imageries*
viii) *Information obtained from interview/enquiry to the experienced engineers/specialists of related speciality, specifically who belonged to the design firms and construction contractors.*
iv) *Documents of previous projects of not only bridge but also Dam, River training, High buildings, Slope protection, Disaster restoration, etc.*

2.3.5.2 The most reliable data is the results of site survey. However, methods, quantity and details of specification shall minimized utilizing the data collected with budget constraint.

2.3.5.3 Due to physical difficulties for site investigation on steep slopes or in deep valleys, the cost of site investigation tends to be more expensive than that in plain terrain. However, the collected data by other method apart from site investigation is also limited due to limited numbers of previous projects and low population density. Therefore, efficiency of the site survey shall be given most priority considering maximizing the quality and quantity and minimizing the cost.

2.3.6 Preliminary Investigation

2.3.6.1 General

(1) All of the preliminary investigations shall be done at the site, as is mentioned in the previous clause 2.3.5.

(2) Investigation and surveys shall be done in this stage is as follows, as is described in the previous clause 2.3.5. Sight amendment has been done in this guideline from IRC SP:54-2018 to apply for the bridges in hilly area. *Italic letters underlined* are the amendments.

Table 2-5: Required Surveys for Feasibility Study

Topographic survey
Hydrological survey
Traffic survey
Sub-soil *(Geological)* investigation *based on boring survey*
Geophysical Survey *based on boring survey*
Survey for land acquisition and resettlement
Environmental survey
Site reconnaissance for construction planning

(3) Planning of each preliminary investigation shall be carefully made to reduce additional site survey to execute preliminary design. Since the condition of each location tends to be not unique and differ within the region in hilly area, the closer attention shall be paid in planning the preliminary investigation for bridge in hilly area. However, this guideline does not restrict executing additional investigations at site to confirm the site conditions for preliminary design. Additional investigation shall be done when the need is justified.

2.3.6.2 Topographic Survey

(1) Topographic survey shall be executed at site to confirm the site conditions, such as terrain (contour, slope, river, etc.), crossing objects (rivers, canals, ditches, roads, railways, electric lines, etc.), and land use (buildings, farm, etc.), vegetation, etc.

(2) Aerial photos can be utilized and surface survey results shall be collected to adjust the aerial photos. This could save cost and time for the investigation.

(3) For minor bridges, it required 1:500 scales for general view. The quality of the survey shall satisfy this need in primary design.

(4) For major bridges, the scale of plan and profile of general view will be smaller, i.e. for the bridge of 500m long, 1:2000 is prepared to fit A3 size paper. However, to study the location of substructure, topographic map of at least 1:500 or bigger scale shall be prepared. The quality of the survey shall satisfy this need in primary design.

(5) The terrain of the region in hilly area tends to have numerous discontinuous surfaces, such as cliffs, exposed rocks, depression, etc. These discontinuous surfaces shall be surveyed carefully and shown in the map.

(6) The area of survey shall be extended when the site is far from the existing road and the access road shall be planned. Especially the surface survey shall be done with great care when the access road shall cross the crossing objects (rivers, canals, ditches, roads, railways, electric lines, etc.).

(7) Topographic survey shall cover all the alternative site of the bridge.

2.3.6.3 Hydrological Survey

(1) Hydrological survey at site shall be executed to confirm the hydrological conditions, such as cross-section, gradient, terrain which influence the river flow (drops, rapids, bends, huge rocks located in the streamline, etc.), structures which influence the river flow (darns, barrages, weirs, spurs, revetment, dikes, etc.), etc. to determine the hydrological conditions for preliminary design.

(2) Surface survey shall be carefully done at the location where the river cross-section changes discontinuously, such as drops, huge rocks, darns, barrages, weirs etc.

(3) Surface soil of riverbed, especially upstream of the alternative site of bridge, shall be investigated to understand the characteristics of debris in flood.

(4) Surface soil of riverbed at alternative sites for bridge shall be done to provide the conditions to study the scour depth of substructure. Bearing layer for bridges in hilly area tends to locate at the shallow depth below the surface ground and spread foundation is likely to be selected. Therefore, misunderstanding of the scour at the site can leave serious damage, collapse of bridge in most of the times, after the bridge opening.

(5) Vegetation is also affects to flood flow including the debris. Vegetation of the upstream of the catchment area shall be investigated. Size of fallen trees defines the vertical clearance above HWL.

(6) The area of survey shall be extended when the access road crosses the river at different crossing location far from the bridge cross section.

(7) Hydrological survey shall cover all the related rivers of alternative site of the bridge.

2.3.6.4 Traffic Survey

(1) Traffic survey shall be conducted to study the traffic projection of the bridge and define the number of lanes of the bridge. The former is for economic analysis and the latter is for preliminary design.

(2) If the bridge is planned to be a new crossing bridge for the major river at site, not only the existing bridge but also other crossing facility such as ferry boat shall be the location of traffic survey. There are many locations as candidate site for new crossing location of major rivers in the hilly area in India. Many of these don't even have ferry boat for heavy vehicles. Therefore, only counting the traffic volume of the existing crossing ferry does not represent the current needs for bridge implementation. Experienced traffic engineer shall be the one to plan and supervise the traffic survey in this case.

(3) If the survey is done in planning of the entire highway/expressway, specific survey for bridge planning is not necessary. The result of the survey in highway/expressway planning shall be applied.

2.3.6.5 Sub-soil Investigation

(1) Sub-soil investigation shall be conducted by boring survey at site. The location of bore holes shall be decided with great care, since the bearing layer tends to be not in unique depth. It has gradient in most of the cases. Therefore, it shall be done along the alignment at more than three points. It is recommended to conducts the boring survey at the possible location of substructure.

(2) Sub-soil investigation can be conducted by observing the cut surface of slope or surfaces of cliff during site reconnaissance. Gradient and direction of sub-soil layers can be observed in this investigation. The slope has high risk of land slide when the gradient is steep. Alignment of approach shall be studied to detour these areas or slope protection countermeasure shall be studied in preliminary design. Degree of weathering of cut surface of slope or surfaces of cliff shall be investigated at the same time to understand the risk level of land slide.

(3) There are some areas in wide valley with river sediment relatively new. It often forms marsh land and prone to face consolidation when load is placed on top. It also can be urban area/farmland/rangeland if the sedimentation stage is at further stage. Consolidation test shall be conducted in the laboratory using the samples of collected by boring survey.

(4) Sub-soil investigation shall cover all the alternative site of the bridge.

2.3.6.6 Geophysical Investigation

(1) Geophysical investigation shall be conducted by boring survey at site to reveal depth of bedrock, type of rock/bedrock, layers and fractures in rock. The rock layers mostly have gradient and the depth from the ground surface differs by the points. Several boring survey shall be done when the gradient and depth differs significantly and randomly.

(2) Appropriate laboratory tests shall be executed to the samples of rock layers collected by boring survey. Bedrock shall be selected carefully to sustain the substructure and bridge. When the sedimentation layers is on the rock layer in river flow cross section, the scouring analysis shall be done as is describes in Clause 2.3.6.3.

2.3.6.7 Survey for land acquisition and resettlement (added)

(1) Quantity of land acquisition (area) and resettlement (number of inhabitants, buildings, etc.) shall be minimized basically in infrastructure implementation project. Land use of the alternative site shall be investigated.

(2) Important facilities such as religious temples, schools, hospitals, graveyards, etc. shall be detoured. The existence and the importance of them shall be investigated and checked in site reconnaissance.

2.3.6.8 Environmental survey (added)

(1) Site investigation of rare and endangered species of flora and fauna shall be conducted when there is related information of them.

(2) Site reconnaissance to determine the negative impact by bridge implementation including the time during the construction and after the opening shall be investigated.

(3) Specific procedure shall be regulated in implementing major bridges, related regulations including the regulation issued by other authorities, i.e. Ministry of Environment, Forest, and Climate Change, it shall be followed in environmental site investigation.

2.3.6.9 Site reconnaissance for construction planning (added)

(1) Construction condition is usually very severe in the bridge construction in hilly area. Bridges planned to cross deep valley tends to have substructures on the steep slope. There is very limited flat land in these locations, and cut and fill/temporary jetty is needed. However, if the slope is steep, it is not technically feasible in some cases or requires high cost, impacts the environment very negatively. Therefore availability of construction yard shall be investigated in site reconnaissance.

(2) Securing access route to location of substructures to transport construction materials and machinery is very important factor in deciding the site, alignment of the bridge and location of substructures. Abutments can be accessed from the main alignment, but not piers. If the slope is steep, it is not technically feasible in some cases, or access road will be long and expensive and it will harm the environment severely. Therefore, the minimizing the access route is one of important policy in construction planning. The conditions for constructing access route to piers shall be collected in site reconnaissance.

(3) If the crossing river is wide, construction condition of pier in riverbed/bank shall be checked in site reconnaissance. The existence and size of bolder restricts construction method of foundation. Seasonal change of river discharge restricts the timing of construction.

2.3.7 Preliminary Engineering

2.3.7.1 Design Philosophy

(1) Design philosophy for the all the personnel involved in implementing new bridge projects, mainly engineers/specialists including project owner and project managers, is very basic and fundamental principal during their total involvement of the project.

(2) Bridges treated in this guideline form each section of road, such as highways/expressways. Therefore, the characteristics of the highway/expressway define the majority of design philosophy of the bridge in implementing/engineering point of view.

(3) Road category, importance, and its specific function request the bridges to be uniform with the other sections of highways/expressways. If the bridge is a part of national arterial expressway network, it has to be sufficient to meet the requirement as a member of national arterial expressway. The detailed design conditions are derived from the requirements of road, mainly the strength and durability (How strong, and how long to use). The characteristics of highways/expressways are as follow,

Table 2-6: Characteristics of Highways/Expressways which Needs to be Defined

i) Road Category
ii) Classification (national/, state, district, village, private, etc.).
iii) Importance (arterial/trunk or feeder, urban of rural, intercity or city to rural area, etc.)
iv) Terrain (plain, rolling, mountainous, steep)
v) Usage/Location (airport road, harbour road, mine road, etc.)
vi) Special function (emergency route, natural disaster/terrorism/conflict, etc.)

note) This guideline treats only the bridges in mountainous and steep terrains.

(4) Another important issue which defines the design philosophy is the characteristics of the bridge itself. It is often that the major bridges become famous and becomes the landmark of the region. Design philosophy shall be different with the ordinary bridge which just has the purpose of sustaining the road transportation. Aesthetic point of view shall be given higher priority so that the local residents can be proud of.

(5) One more important issue for major bridges is that major bridges often challenges the unexperienced conditions. The advanced/state-of-art technology shall be actively applied to cope with the challenging technical issues. Major bridges in hilly area often faces this situation due to their severe natural conditions.

(6) Last issue is to listen to needs of regional community as necessary. Bridges could be the route for important regional ceremony for instance, because it is section of road. Satisfying the local needs is constantly very important design philosophy. Close communication with local authority is highly important.

2.3.7.2 Analysis of Data

(1) Collected data mentioned in previous clauses 2.3.3, 2.3.5, and 2.3.6, requires analysis to determine the design condition of the bridge. As the stage of the implementation is still in early stage, feasibility study (preliminary design), design conditions in this stage also does not require the quality and quantity as same as that in detailed design or construction stage. Literally speaking, every data collected only requires the quality of initial stage; however executing site investigation for bridges in hilly area often needs higher expense both cost and work. Therefore, the quality of the survey shall not be lower considering the repetition in the further stage. Therefore, quantity shall be given to reduce in early stage.

(2) Collected data shall be used to define the main purpose of feasibility study, bridge site. For selection of bridge site, preliminary design/study needs to be done. The results of these designs/studies are used in comparison study. An example of factors for comparison study is as follows. The representative engineer shall ensure that the collected data is sufficient in both quality and quantity.

Table 2-7: Comparison Factor in Selecting Bridge Site

Comparison Factor	As Results of
Structure stability,	Design
Constructability,	Construction planning
Accessibility (to the site),	Construction planning
Procurement easiness	Construction planning
Maintenance easiness	Design
Cost,	Design
Economic efficiency (EIRR),	Economic analysis
Environment,	Environmental impact examination
Aesthetic,	Site reconnaissance
And more specific factor related to the region	Interview, Site reconnaissance

(3) Example of the design/study conditions to be determined by analysing the collected data including the results of site investigation is described in IRC:54:2000 as follows. This guideline will add and modify slightly to apply for the bridges in hilly area and make it more in details. *Italic letters underlined* are the amendments.

Table 2-8: Example of the Design/Study Conditions to be Determined by Analysing the Collected Data

RIVER
i) HFL (High Flood Level)
ii) LWL (Low Water Level)
iii) LBL (Lowest Bed Level)
iv) Erodibility of bed/scour level
v) Design discharge *(flow velocity, cross section)*
I) River Training Plan
II) Deformation of riverbed/channel

SUB-SOIL/GELOGICAL
vi) Likely foundation depth
vii) Safe bearing capacity
viii) Engineering properties of sub-soil
ix) Artesian conditions
x) Settlement characteristics *(including consolidation)*

RIVER/TRAFFIC
xi) Vertical clearances
xii) Horizontal clearance
III) Road/Railway Implementation Plan
IV) Road/Railway improvement Plan

APRROACH (RIVER)
xiii) Free board for approach road

NATURAL ENVIRONMENT
xiv) Severity of environment with reference to corrosion (severe/moderate)
xv) Data pertaining to seismic and wind load
V) Temperature/rain fall intensity/snow depth distribution

PROCUREMENT
xvi) Availability of suitable construction material

COST ESTIMATE
VI) Unit cost of material/labour/works (construction/maintenance)

MAINTENANCE
VII) Required service level/activities
VIII) Special requirement (repainting, snow/ice removal)

ECONOMIC ANALYSIS
IX) Unit cost of benefit calculation (VOC, Time saving, Accident, Emission, etc.)
X) Traffic Volume
XI) Economic statistical and forecast data

ENVIRONMENT PROTECTION
XII) Protection Area
XIII) Types and habitats of Rare and Endanger Species
XIV) Effluent Regulation

LAND ACQUISITION AND RESETTLEMENT
XV) Location of life line facilities including elevated/buried facilities
XVI) Location of facilities to avoid resettlement

SPECIAL REQUIREMENT
xvii) Requirement of model study
XVII) Traditional event in the region

2.3.7.3 Finalization of Design Philosophy

(1) Design philosophy for bridge implementation shall cover large variety of requirement from different fields. It is not just developing a section of highway/expressway which can ignore the presence of other natural and social aspects. Therefore, mutual agreement of design philosophy among stakeholders and related authorities/organizations is fundamental for successful implementation. Sufficient communication and discussion shall be taken for its finalization.

(2) Combined structures (i.e. road and aqueduct, mentioned in Clause 5.7.3, IRC SP:54-2018) are the typical structure which need to fulfil needs of different aspects. There is no difference in design philosophy of combined structures by its located terrain.

(3) Importance of building appropriate design philosophy of subway/underpass is mentioned in in Clause 5.7.3, IRC SP:54-2018. However, this item is excluded from this guideline. It is considered as tunnel in hilly area. It can be referred to ‘Guideline for tunnel on arterial roads in hilly area’ prepared in same project by JICA.

2.3.7.4 Approach Alignment and Siting of Bridges

(1) Sites of candidate crossing location of bridge have been identified in Pre-feasibility study. Alignment of bridge and approach road is studied in the first stage of preliminary design by comparing the alternatives. The followings are the factors to be considered in setting the alternatives. Subchapter 2.2, of POCKET BOOK-2000 shall also be referred.

(2) Considering the high implementation cost of bridge, the length shall be shortened as short as possible. Therefore, securing the clearance requirement which shall be overpassed by the bridge as the design condition is the fundamental condition that the alignment or bridge shall fulfil. To satisfy these two needs, the crossing angle of bridge and crossing facility, such as river, road, railway, canal, life line, etc. shall be close to right angle (90°).

(3) Vertical and horizontal alignment shall be mild as possible with large radii. Moreover, gradient of profile shall not be too steep. These are common requirement in highway alignment design. However, several constraints in hilly area in undulating terrain do not permit ideal alignment in general.

(4) Risk locations of land slide/slope failure shall be avoided for continuous operation and minimization of maintenance cost after the opening. Responsible highway engineer shall corporate with geological engineer to set the alternatives of alignment

(5) Accessibility to locations of substructure construction requires temporally access road. The length of it shall be minimized considering the construction cost and negative impact to the nature. Responsible highway engineer shall corporate with bridge engineer to set the alternatives of alignment.

(6) If the bridge is a major bridge with the longest span length of more than 100m or total bridge length exceeds 300m, minimizing the bridge length shall be given top priority when other condition permits. However, straight alignment is preferable for expressways with high design speed to secure safety for traffic, sufficient sight and improve driving comfortability of drivers. Comprehensive comparison study shall be conducted with alignment alternatives of bridge with contiguous highway/expressway section.

(7) If the bridge is not a major bridge (less than 60m) or the longest span length or total length is not so long (less than 60m), there is no need to set the alternative alignment specifically for bridge. Alignment shall be considered given condition from highway/expressway design result.

(8) The selection procedure as the result of comparison study shall be conducted with the alternatives of bridge structure as the result of preliminary design. It will be described in following clauses.

2.3.7.5 Identification of Feasible Alternative Alignment

(1) Alternative alignments of bridge in each candidate site of crossing location shall be prepared prior to the beginning of preliminary design of bridge, as is mentioned in Clause 5.7.5, IRC SP:54-2018. This condition applies for the project preparation of bridge in any terrain.

(2) It is considered that initial filtering of possible alignment has been done in pre-feasibility study, since the selection of alternative site (crossing location) has been done. More attention shall be paid for constructability, mainly in terms of accessibility to the substructure site, in identifying the alternative site. There are many locations which is extremely difficult/costly to access and secure flat land as construction yard in mountainous and steep terrain. Responsible highway engineer shall corporate with bridge engineer to set the alternatives of alignment.

(3) Extracting very difficult location for substructure construction is fundamental design condition for span arrangement of bridge in hilly area. It indicates that it is necessary to consider span arrangement while identifying the alternative alignment. It is absolutely obvious for bridges in hilly are due to the constraints by terrain and natural condition of the site.

(4) The factors to be considered in identifying the alternative alignment are shown in Subchapter 2.2, of POCKET BOOK-2000.

(5) Apart for construction and structure aspects, volume of resettlement inhabitants and land acquisition shall be minimized. Unavoidable resettlement is cannot be helped in some locations, however it is not easy to find compensation land for the inhabitants in hilly area.

(6) If there is near to the habitats of endangered species, it shall be shifted to the area which does not give negative impacts to the species.

(7) No more than 3 alternatives for each site are needed to be prepared in this stage, since detailed comparison study is executed in further stage. The details are described in following clause 2.3.7.7 and 2.3.7.8.

2.3.7.6 Geometric Design

(1) Alignment of highway/expressway in hilly area does have numerous both horizontal and vertical curves due to its constraints by terrain. There are also sight changes due to variety of road sections, such as fill/cut section, bridges, and tunnels. Therefore securing the drivers' visibility wide enough for their safety is very important in geometric design.

(2) Combination of horizontal and vertical alignment sometimes creates misleading sight to drivers. It could be a cause of vehicle mishandling or speed decrease and derive traffic congestion and accidents. These misleading sights can be exaggerated with the sight change by terrain mentioned in the above sentence in hilly area including mountain bridges.

(3) Vertical sag section shall not be placed in cut section in general. It requires specific drainage treatment, such as drainage pipe, deep ditch, pumping facility, etc. Sag section on bridge requires bigger outlet or many outlets on the surface to drain heavy rain. Moreover, puddles are easily formed in the section and will be a dangerous section for drivers with high velocity. Therefore, location of sag section shall not be placed at the sag section.

(4) If the bridges are located in snow cold region, surface will be frozen in winter. It is difficult to control the vehicle on steep vertical gradient section. Bridge surface tends to be frozen much easier than earthwork section. Avoiding is not possible but reducing the gradient can be done from planning stage.

2.3.7.7 Span Configuration and Type of Superstructure Systems

(1) Economically efficient span length for average height is around 25 to 40m in general. However, new bridges construction site in hilly area has its construction constraints especially for access to substructure site. It significates that substructure is not feasible to construct in many location within bridge section. Following areas shall be avoided for substructure, very steep slope, cliff, river section with large size debris, unstable river channel, and land slide/slope collapse risk area, adjacent to existing structure. These areas have strict conditions for span configuration in bridge planning in hilly area. It is recommended to make the list of restriction points/condition for further study.

(2) After extracting restricted areas for substructures, there are candidate locations for substructures along the alignment. Each combination pattern of substructure placement is candidate of span configuration for the bridge. First filtering shall be done by extracting very short and long span length compared to other candidates, ill-balanced span configurations. After the first filtering, there are initial alternatives of span arrangement.

(3) Next step is to apply suitable superstructure type and set the substructure type and height. Piers are mostly RCC wall type or column type, and abutments are mostly wall type. If the superstructure is wide, 4 lanes or more, wall type exceeds column type. This shall be identified in next filtering step. Superstructure type is demonstrated in Clause 5.7.7, IRC SP:54-2018. This guideline will add and modify slightly to apply for the bridges in hilly area and make it more in details. *Italic letters underlined* are the amendments.

Table 2-9: Suitable Type of Superstructure by Span Length

	Type of superstructure	Span length
RCC		
(i)	RCC single or multiple boxes	1.5 to 15 m
(ii)	Simply supported RCC slabs	3 to 10 m
(iii)	Simply supported RCC T-Beam	10 to 25 m
(v)	Simply supported RCC voided slabs	10 to 15 m
(vi)	Continuous RCC voided slabs	10 to 20 m
(viii)	RCC box sections; simply supported/balanced cantilever continuous	25 to 50 m
PSC		
(iv)	Simply supported PSC girder bridges *(post-tensioned)*	25 to 45 m
	Simply supported PSC girder bridges (pre-tensioned)	*5 to 25 m*
	Simply supported PSC girder bridges (Bi-Prestressing)	*25 to 50 m*
	Simply supported PSC composite girder bridges	*40 to 60 m*
(vii)	Continuous PSC voided slabs	15 to 30 m
(ix)	PSC Box sections, simply supported~~/balanced cantilever~~	35 to 75 m
	PSC Box sections, span-by-span pre-cast segmental method	*50 to 100 m*
(x)	PSC Box sections, cantilever construction/continuous	50 to 150 m
	Rigid frame bridges, simply supported	*30 to 55 m*
	Rigid frame bridges, cantilever erection method	*40 to 140 m*
	Extradosed bridges, simply supported	*50 to 100 m*
	Extradosed bridges, cantilever erection method	*100 to 200 m*
	Cable stayed bridges	*100 to 250 m*
STEEL		
	H-beam girder bridges	*10 to 25 m*
	Plate girder bridges	*25 to 60 m*
	Plate girder bridges with Steel Orthotropic Deck	*30 to 70 m*
	Box girder bridges	*30 to 80 m*
	Box girder bridges with Steel Orthotropic Deck	*40 to 150 m*
	Arch bridges, (Langer)	*50 to 120 m*
	Arch bridges, (Lohse)	*70 to 180 m*
	Arch bridges, (Network-tied/Nielsen-Lohse)	*50 to 170 m*
	Steel orthotropic slab	*30 to 150 m*
(xi)	Cable stayed bridges	200 to 500 m
(xii)	Suspension bridges	500m onwards
STEEL-CONCRETE COMPOSITE		
	Corrugated Steel Web, T-shaped girder	*20 to 50 m*
	Corrugated Steel Web, box girder	*40 to 150m*
	Composite Truss, box girder, outer-cabled	*30 to 100 m*
	Composite Truss, box girder, extra-doused	*100 to 180 m*
	Mixed Steel/PSC, integral (rigid frame)	*50 to 90 m*
	Mixed Steel/PSC, extra-doused	*200 to 250m*

(4) Locations of abutments are identified by the height difference between alignment and the existing ground level. Securing the construction yard at the point is required in this step.

(5) Rough estimation of foundation type is identified for each substructure, both abutments and piers, in each alternative referring sub-soil investigation results. Type of foundation is among spread, pile, and well in most of the cases. It can be temporarily identified foundation because it shall be compared and selected in detailed comparison study for foundation.

(6) Rough general view is prepared for each alternative. And adjustment of span length is done to set the alternatives for identifying alternatives for comparison study for bridge site. No more than 3 alternatives for each alignment are needed to be prepared in this stage, since detailed comparison study is executed in further stage. The details are described in following clause 2.3.7.15.

(7) Span configuration of the bridges in hilly area is summarized as follows,

Table 2-10: Span Configuration of the Bridges in Hilly Area

i) Extract restricted areas of substructure construction
ii) Make combinations of span length with in the bridge, extract ill-balanced combinations
iii) Apply suitable superstructure types and set height of substructures
iv) Set location of abutments
v) Estimate foundation type
vi) Prepare rough general views for each alternative for each candidate site

2.3.7.8 Comparison Study for Bridge Site and Alignment (added)

(1) The site and alignment has not been fixed yet at this stage. The comparison study shall be done for every general views prepared in span configuration mentioned in previous clause 2.3.7.7. The method can be done in hybrid manner of quantitative and qualitative. This comparison study will make the complicated conditions caused by unique characteristics of each alternative site simple. Generally speaking, the steeper the terrain is the severer the complicacy of this comparison.

(2) Quantitative factors are selected among followings, bridges length, surface area, roughly estimated cost, distance/travel time shortened after the construction for users.

(3) Qualitative factors are selected among followings, structure stability, constructability, accessibility, procurement easiness, maintenance easiness, impact to the flood/river flow, impact to the riverbed/bank, risk of natural disaster, environmental impact, scale of resettlement, traffic condition improvement, and more local factors. These factors can be quantified by segments of level for easier comparison.

(4) There are factors for comparison for river bridges shown in Subchapter 2.2, of POCKET BOOK-2000.

(5) Comparison results of each factor studied shall be summarized in a table for comprehensive comparison. Each factor shall be given priority and weight index to sum up the total evaluation value for each alternative. The site which has top evaluated alternative is the site for bridge construction and further processes will be followed.

(6) This process shall be done by collaboration among bridge, highway, geological, and hydrological engineers. Apart from those engineers, advises shall be given from traffic, environmental, and material engineers and resettlement specialist. Comprehensive judgement is fundamental to select appropriate site and alignment.

(7) This process can be omitted if the bridge is minor and forms a part of highway/expressway project. Bridge site is considered as give condition from alignment of highway/expressway.

2.3.7.9 Aesthetic Considerations

(1) Implementation of bridges in hilly area is likely to be done in natural environment with vegetation, natural slopes, and rivers surrounded or nearby. Therefore, vision of the bridge shall be harmonic to the natural scenery.

(2) Major bridge will have significant presence at the site and region. It would be visible from surrounding area and long distance. However, it shall not change the characteristics of landscape at the site. Therefore, close attention shall be payed not to attract too much attention and maintain the aesthetics of the landscape of the site. Designers/engineers shall cooperate to give their effort in designing the bridge which improves the landscape of the site.

(3) It is recommended to create visual image of the bridge after the implementation at the site to compare the landscape of the bridge, such as computer graphics or photo montage to compare the aesthetic aspect. If the steel bridge is selected visual image can be utilized for colouring selection. However, colour can be discussed and selected in further stage.

2.3.7.10 Preliminary Design

(1) Technical feasibility and the most adequate bridge structure needs to be defined by this preliminary design step. Preliminary structural calculation shall be done to secure the technical feasibility. Following items shall be set to run the calculation.

Table 2-11: Items Required for Structural Calculation in Preliminary Design

i) Span arrangement
ii) Type and scale of superstructure
iii) Type and scale of substructure
iv) Type and scale of foundation

(2) Before starting the preliminary design steps, design condition shall be summarized considering the steps conducted mentioned above. The conditions are basically same for bridges in any terrain. However, some features for bridges in hilly area are specifically mentioned below in *italic underlined letters*.

Table 2-12: Design Conditions Required for Structural Calculation in Preliminary Design

LOAD (Source: IRC:5-2015, Standard Specification and Codes of Practice for Road Bridges, Section II)

1)	Dead Load	G
2)	Live Load	Q
3)	Snow Load Snow load shall be carefully examined in Himalayan Mountain States, since the accumulation can exceed 0.50m in some region.	Gs
4)	Impact factor on vehicular live load	Qim
5)	Impact due to floating bodies or vessels as the case may be This load can be basically extracted in bridges in hilly area	Fim
6)	Vehicle collision load	Vc
7)	Wind load Wind velocity shall be examined carefully in steep slope and deep valleys, since the wind at site varies considerable from the record of observatory. It also blows 3-dimentionally due to complicated terrain feature.	W
8)	Water current Water current shall be examined carefully in catchment area with deep valleys, since the rain reaches to the site more rapidly than in plain area with more discharge.	Fwc
9)	Longitudinal forces caused by tractive effort of vehicles or by braking of vehicles and/or those caused by restraint of movement of free bearings by friction or deformation	Fa/Fb/Ff
10)	Centrifugal force	Fcf
11)	Buoyancy This load can be basically extracted in bridges in hilly area except in the	Gb
12)	Earth pressure including live load surcharge, if any	Fep
13)	Temperature effects Minimum temperature shall be carefully examined in bridges in hilly area with high altitude.	Fte
14)	Deformation effects	Fd

15)	Secondary effects	Fs
16)	Erection effects	Fer
17)	Seismic force	Feq
18)	Wave pressure This load can be basically extracted in bridges in hilly area	Fwp
19)	Grade effect Grade effect shall be carefully *examined since the bridges in hilly area tends to have the smaller radius and steeper longitudinal gradient. Lower road category roads shall be specifically paid attention.*	Ge

RIVER

1) Cross section of river crossing location
2) HWL
3) Vertical clearance above HWL
4) Return period of rain fall to be considered
5) Discharge and velocity of the flood flow
6) Surface and sub-soil data of riverbed (for scouring study)
7) Debris data (for study of protection of pier)
8) Free board above horizontal clearance (avoid debris crush to superstructure)
9) River training plan (future river cross section)

ROAD/RAILWAY

1) Vertical clearance
2) Horizontal clearance
3) Free board above horizontal clearance (for future overlay)
4) Future widening plan

SUB-SOIL/GIOLOGICAL

1) Bearing Layer and its bearing capacity
2) Layer with possibility of consolidation and its settlement
3) Land slide/slope collapse risk area

CLIMATE

1) Officially recorded precipitation data
2) Officially recorded wind data
3) Return period to be considered (for flood, wind, and surface drainage, moreover snow in some bridges)

CONTRUCTION

1) Restricted area for substructure construction
2) Possible access route for substructure construction

OTHER

1) Restricted/protection area for natural environment
2) Properties should not be relocated
3) The existing lifeline and future plans
4) Land use of the catchment area

(3) There shall be the selected site of bridge and alignment after the step described in clause 2.3.7.8. Selection of the span arrangement and type of superstructure shall be done in this process to define the general features of the bridge.

(4) First step in this process is to accommodate the span length next to each other to minimize the span length group in multi-span bridge. Length of each span shall be grouped to simplify the structure of the bridge. It means that number of group shall be as less as possible, i.e. the most of the viaduct in plain terrain has unique span length for all the spans. And also the span next each other shall be within the same span length group as much as possible. Number of the group of span length shall not be more than 2 ideally in the same bridge. If the span next each other has different span length, the difference shall be minimized as possible. However, strong attention needs to be paid not to place the substructures in restricted area. If the bridge is single span bridge, this accommodation is not necessary.

Table 2-13: Example of Setting the Alternatives for Comparison Study of Span Arrangement.

I.E.: a bridge of 4 spans,

From the possible location of substructure was, span 1=35m, span 2=50m, span 3=25m, span 4=40m, total 150m.

It can be accommodated as 2@75m, 85+65m, 35+2@57.5m, 2@55+40m, 3@50m, or 4@37.5m. (with no substructure construction constraints, if so omit from alternatives)

It cannot be so simple as above at the site of bridge in hilly area with restricted construction area for substructure.

Bad arrangement is 35+45+35+45m, 35+80+35m, 35+105m or 110+40m

(5) Possible combinations of accommodated span arrangement and type of superstructure of selected site and alignment shall be longlisted for comparison study for bridge structure. The table in Clause 2.3.7.7 (or the following clause (6)) shall be used to this matching for longlisting. Though there are very large numbers of possible combination in some sites, EVERY combination shall be listed in this longlist. Listed combinations are ready to prepare the bridge cross-section with the width of highways/expressways. Every section of each superstructure shall have its cross-section prepared. Numbers of girders or width of box girder can be set temporally referring the past projects of same types of superstructure.

(6) Height of superstructure will be automatically set from the span length, using the most efficient height/length ratio. It is shown in the following table.

Table 2-14: Suitable Height/Span Length by Types of Superstructure

Type of superstructure		Span length	Height/Length
RCC			
(i)	RCC single or multiple boxes	1.5 to 15 m	1/13
(ii)	Simply supported RCC slabs	3 to 10 m	1/13
(iii)	Simply supported RCC T-Beam	10 to 25 m	1/13
(v)	Simply supported RCC voided slabs	10 to 15 m	1/18
(vi)	Continuous RCC voided slabs	10 to 20 m	1/18
(viii)	RCC box sections; simply supported/balanced cantilever continuous	25 to 50 m	1/13
PSC			
(iv)	Simply supported PSC girder bridges (post-tensioned)	25 to 45 m	1/15
	Simply supported PSC girder bridges (pre-tensioned)	5 to 25 m	1/15
	Simply supported PSC girder bridges (Bi-Prestressing)	25 to 50 m	1/30
	Simply supported PSC composite girder bridges	40 to 60 m	1/15, 1/17(1)
(vii)	Continuous PSC voided slabs	15 to 30 m	1/22
(ix)	PSC Box sections, simply supported/balanced cantilever	35 to 75 m	1/18
	PSC Box sections, span-by-span pre-cast segmental method	50 to 100 m	1/18
(x)	PSC Box sections, cantilever construction/continuous	75 to 150 m	1/25
	Rigid frame bridges, simply supported	30 to 55 m	1/22
	Rigid frame bridges, cantilever erection method	40 to 140 m	1/25
	Extradosed bridges, simply supported	50 to 100 m	1/30
	Extradosed bridges, cantilever erection method	100 to 200 m	1/50
	Cable stayed bridges	100 to 250 m	---
STEEL			
	H-beam girder bridges	10 to 25 m	700-900(2)
	Plate girder bridges	25 to 60 m	1/15

	Plate girder bridges with o Steel Orthotropic Deck	30 to 70 m	1/30
	Box girder bridges	30 to 80 m	1/19
	Box girder bridges with Steel Orthotropic Deck k	40 to 150 m	1/30
	Arch bridges, (Langer)	50 to 120 m	1/6
	Arch bridges, (Lohse)	70 to 180 m	1/6
	Arch bridges, (Network-tied/Nielsen-Lohse)	50 to 170 m	1/6
(xi)	Cable stayed bridges	200 to 500 m	---
(xii)	Suspension bridges	500m onwards	---
STEEL-CONCRETE COMPOSITE			
	Corrugated Steel Web, T-shaped girder	20 to 50 m	1/20
	Corrugated Steel Web, box girder	40 to 150m	1/38
	Composite Truss, box girder, outer-cabled	30 to 100 m	1/14
	Composite Truss, box girder, extra-doused	100 to 180 m	1/55
	Mixed Steel/PSC, integral (rigid frame)	50 to 90 m	---
	Mixed Steel/PSC, extra-doused	200 to 250m	---

(1) I-shape: 1/15, U-shape: 1/17, (2) Product size is whether 700, 800, or 900mm

(7) Second step is to define substructure type and height. Height can be set from the general view and superstructure height already set previously. Type of piers can be set by the superstructure width. If it exceeds 2 lanes, wall type shall be selected. If it is less than 2 lanes, column type can also be other alternative. However, in this step wall type is sufficient. Comparison can be done in further stage. Types of abutment can be set as wall type. The higher the abutment is, the shorter the bridge is. Therefore it is basically better to have higher wall to save the construction cost. However, it can be set to 5 to 7m temporally.

(8) Third step is to set the type of foundation. It is set by the depth of bearing layer. If the top surface of bearing layer locates less than 7m, spread foundation shall be set. If it located 7m deep or deeper (assuming that cover of footing exceeds 5m), pile foundation shall be selected. While there is construction constraints to secure flat land to place construction machinery, Caisson Foundation shall be set. All substructures have footings at the bottom of wall and the footing has minimum cover thickness as 1m. Length of piles and caisson can be set from the bottom surface of the footing to deeper level than top surface of the bearing layer. Pile/caisson shall be grounded more than its diameter depth to the bearing layer. Number and the diameter of foundation which has piles or caissons can be temporally set for preliminary calculation referring past project experience. If the foundation type selected cannot be justified the selection procedure, comparison study of possible alternative types shall be compared. However if the construction constraints are sever, the alternative is omitted from the comparison study. Main comparison factor is construction cost. Other factors can be the risk to natural disaster, and constructability.

(9) Fourth step is preparing general view (profile and plan) for the alternatives longlisted in previous clause (4). The profile shall have sub-soil data, and bearing layer shall be demonstrated clearly. Plan has to have the contour in details.

(10) Fifth step is to filter the longlisted alternatives to approximately 3 alternatives. Comparison factors are cost (construction and maintenance), constructability, and easiness of maintenance, risk to natural disaster, natural/social environmental impact, and landscape. The results of comparison of each factor shall be quantified and sum upped with weight applied as total evaluation. Cost shall be put heaviest weigh since it is the most important factor.

(11) Next step is preliminary structural calculation of 3 filtered alternatives to secure the technical feasibility. The procedure is same for most of the bridges. However, for confirmation it is summarized as follows,

Table 2-15: Necessary Combination of Structures for Preliminary Structural Calculation

i) filtered superstructures in step (10) with the type and height set in (5) and (6)
ii) filtered substructure in step (10) with the type and scale set in (7)
iii) filtered foundation in step (10) with the type and scale set in (8)

(12) In case of abutment, the superstructure can be shortened if the abutment is higher. However, it requires high abutments and wide footing and construction site. Most of the bridges in hilly area have its abutments in slopes which is difficult to secure the flat land. There is a trade-off issue between superstructure length and abutment height which needs to be solved. Comparison shall be made to minimize the implementation cost by optimizing the span length of the spans of abutments and height of abutments.

(13) After the preliminary structural calculation, quantity is calculated for cost estimate. This step is basically the same for bridges in hilly area.

2.3.7.11 Construction Planning (added)

(1) There are many occasions in new implementation of the bridge of highways/expressways in hilly area that there isn't any the existing access route to the construction sites. Specifically the construction sites for piers require temporally access route for construction machinery/materials to access.

(2) For all the substructures, abutments/piers, it requires certain width of flat land for the construction. Generally speaking, caisson foundation requires the least area and pile foundation requires the most due to the necessity of considerable size of drilling machinery. Spread foundation depends on its height and the load which the substructure has to bear. Therefore, not only how to access the site but also how to secure the flat land needs to be considered carefully to decide the location of sites for substructure construction.

(3) Construction site for superstructure needs to be examined carefully to minimize the impact to the natural environment. For girder bridges, the area behind the abutments shall be utilized to minimize the use of natural land of cutting area. If the superstructure can be constructed by balanced cantilever, it is recommended to study it as an alternative of the construction method because it is normally requires less construction site.

(4) Aggregates crusher plant and cement concrete batching plant are usually placed newly for bridge construction in hilly areas. The former nearby the borrow pit, and the latter nearby the bridge construction site in general. It also needs to be planned carefully to minimize the impact to the natural environment.

(5) Most of the urban construction sites do not require considering above mentioned construction condition and planning in project preparation stage. However, it is usual for the bridge construction in hilly area. Not only the construction planning but also the cost for temporally structure shall be taken in consideration in project preparation for bridges in hilly area. Construction cost, specifically for the temporary facilities, of bridges in hilly area has strong tendency to increase compared to the site in plain area.

(6) Construction schedule shall be prepared in this step. In case of bridges which crosses river, the construction shall consider the rain/dry season. Substructure/foundation construction is often restricted in rain season. The river flow depth as well as precipitation shall be studied in planning the construction schedule. The rivers in hilly area has very obvious characteristics of seasonal change, close communication with hydrological engineer is indispensable in this step.

2.3.7.12 General Arrangement Drawings

(1) As it is already mentioned various times above, profile and plan are prepared for all the alternatives. In addition to that, general view of superstructure(s), abutments, and piers shall be prepared for the 3 alternatives for which the preliminary design has been done.

(2) Approach road and river revetment shall be demonstrated in bridge plan and scale of them shall be noted for cost estimate.

(3) Construction plan of the 3 alternatives shall be demonstrated roughly for cost estimate. Plan of access road and construction yard in the site shall be prepared.

(4) Since construction cost in hilly area tends to be higher than that in plain area, this planning effects the total implementation cost of the project. Copying the results from other project with small modification has high risk of budget shortage which leads to the project termination. It is the worst story of the project implementation which all of the stakeholders shall collaborate to avoid. Construction planning by experienced engineer is indispensable in projects in hilly area.

2.3.7.13 Preliminary Cost Estimates

(1) Construction cost, as well as implementation cost is always the most important factor to judge/justify the project's implementation. It is the same for the bridges in hilly area. However, the cost per unit surface square meter varies significantly by site even with the similar length and same type of superstructure between the bridges in hilly area and plain area.

(2) Apart from the cost until the opening of the bridge, operation, maintenance and rehabilitation cost shall also be taken in to consideration. The cost in comparison studies in various steps shall be done by the life cycle cost which includes not only the construction cost but also the maintenance/rehabilitation cost (hereinafter called LCC).

(3) There are 4 steps which this guideline treated the cost estimates.

Table 2-16: Steps of Cost Estimate in Feasibility Study

i) in comparison study for bridge site and alignment (Clause 2.3.7.8 (2))
ii) in filtering longlisted span arrangement alternatives to 3 alternatives. (Clause 2.3.7.10 (10))
iii) in selecting height of abutment before preliminary calculation (Clause 2.3.7.10 (12))
iv) in comparison study in preliminary design (Clause 2.3.7.12), this clause.

(4) The cost estimates in this step i), is to select the bridge site and its alignment along with the approach road connected to it. Since there is not any bill of quantity based on the design result, it cannot be helped that the estimation is roughly calculated. However, it shall be taken in account in calculation that the site condition affects to the cost. The cost items to be estimated shall be as follows, rehabilitation cost is mainly repainting of steel superstructure.

Table 2-17: Items of Cost Estimate in Step i) of Feasibility Study

i) superstructure construction cost
ii) substructure construction cost
iii) foundation construction cost
iv) temporary works cost
v) operation, maintenance, and rehabilitation cost
vi) approach road construction and river protection work cost

(5) The cost estimates in this step ii), is to filter numerous alternatives to 3 alternatives for conducting preliminary engineering. The procedure and the details of this step is same with (4) above except the cost of approach road construction is extracted since it is same for all the alternatives.

Table 2-18: Items of Cost Estimate in Step ii) of Feasibility Study

i) superstructure construction cost
ii) substructure construction cost
iii) foundation construction cost
iv) temporary works cost
v) operation, maintenance, and rehabilitation cost
vi) river protection works cost

(6) Results of cost estimates of some alternatives, alternatives of selected sites and alignment, have already been calculated in the comparison study of selecting bridge site and its alignment in Paragraph 2.3.7.8 (2) (mentioned in step i) in this clause).

(7) The cost estimates in this step iii), is to identify the abutment height and superstructure length of the span adjacent to abutment.

Table 2-19: Items of Cost Estimate in Step iii) of Feasibility Study

i) superstructure construction cost
ii) abutment construction cost
iii) foundation construction cost
iv) temporary works cost

(8) Operation, maintenance and rehabilitation cost and approach road construction and river protection works, can be extracted in this step since it only differs slightly by alternatives.

(9) The cost estimates in the step iv), is to select the optimum span arrangement and type of superstructure. It shall be based on bill of quantity prepared in preliminary design.

Table 2-20: Items of Cost Estimate in Step iv) of Feasibility Study

i) superstructure construction cost
ii) substructure construction cost
iii) foundation construction cost
iv) temporary works cost
v) operation, maintenance, and rehabilitation cost
vi) river protection works cost

(10) To compare the implementation cost of the alternatives, following cost items shall be added to all the alternatives, approach road construction cost, environmental countermeasure cost, land acquisition cost, contingency cost, price escalation cost, and supervision cost. However, it is not necessary for comparison study since these cost items differs slightly by alternatives.

(11) For the cost estimates of temporally works, the following items shall be included,

Table 2-21: Items of Cost Estimate of Temporary Works in Feasibility Study

1) erection of superstructure
2) access road: cut and fill, pavement,
3) construction yard: cut and fill, jetty in the river, working platform,
4) crusher plant: yard development, machinery, running cost, etc.
5) batching plant for cement concrete: yard development, machinery, running cost, etc.
6) stock yard; yard development
7) site offices: yard development, building, running cost, etc.
8) temporally residence for labour: yard development, building, running cost, etc., and more if any.

(12) If the 3 alternatives have some location of piers, the construction temporary cost will be very similar, only 1) erection of superstructure differs by alternatives. However, all the items shall be calculated to avoid misunderstanding of decision makers/readers of the report. Item 1) to 3) usually differs considerably by alternatives if they have different span arrangement. It shall be studied carefully from the data collected especially by site investigation and reconnaissance, since it could be the critical comparison factor for comparison study for the bridges in hilly area. The responsible engineer for this step shall be well experienced in construction works in hilly area.

2.3.7.14 Initial Environmental Examination

(1) There is Clause 5.7.12. in IRC SP:54-2018 describing the initial environmental examination. It covers both natural and social aspects of environment. In this guideline, these 2 aspects are separated to 2 clauses. Firstly, the natural aspect is treated.

(2) There are 2 aspects in natural environmental impact. First is the permanent impact, and the other is temporally. Since the nature in the hilly area is often very fragile against construction projects, both aspects shall be taken seriously in project preparation of bridges in hilly area.

(3) Permanent impacts are mostly derived from 2 construction works, one is river flow change by piers or riverbanks, and the other is separation of habitats by approach road. The former gives negative impacts to the species which exist in downstream of new construction site. If it is affected negatively by the change, it would fade away and the population/habitat area of specific species would disappear/decrease significantly. Even the smoothing of river flow by constructing of river banks can harm the specific species. The latter leads to separate the habitat area of specific species. It can also decrease the population/habitat area of them.

(4) Temporary impacts are mostly due to increase of mad/sediment flow during the construction period. Sometimes due to malmangement of contractor, chemicals/construction materials flow out from construction yard. These give negative impacts to natural environment until the end of the construction, and even leave negative impacts until the natural condition recovers from the harm. If there exist rare or endangered species, it could harm them fatally and it could end up extinguish them. This is one of the most crucial harm in construction project. If the alternative site location is very near to the habitats of rare or endangered species, it shall be considered extraction from comparison study.

(5) Chemicals flowed away from the construction site can give negative impact to inhabitants health/life. This shall be strictly prohibited and the contractor shall control the disposal both liquid and solid. Each chemical which is used for contraction has potential to harm the environment, and is strictly regulated by relevant authority. The regulation shall be obliged throughout the construction period.

2.3.7.15 Initial Land Acquisition and Resettlement Study (added)

(1) There are residential areas even in hilly terrain and needs of highways/expressway implementation including bridges are strong in some regions. In this case, the number of people who needs to resettle shall be minimized. Also area of land acquisition shall be minimized. It needs careful consideration when it is difficult to prepare the compensation land nearby

(2) Public facilities, such as hospital, school, temples, church, and etc., shall be avoided as a resettlement object. It is also applicable to historical monument as is written in Clause 5.7.12 IRC SP:54-2018.

(3) Other impacts to inhabitants nearby, such as noise, vibration, air pollution and etc., is described in Clause 5.7.12 IRC SP:54-2018.

2.3.7.16 Selecting Bridge Type for Basic/detailed Design (added)

(1) Comparison study shall be conducted for 3 alternatives. The factors are LCC, constructability, easiness of maintenance, risk to natural disaster, natural/social environmental impact, and landscape. It is basically the same in the study already conducted in Clause 2.3.7.10 but more in details, specifically the LCC. The results of comparison of each factor shall be quantified and sum upped with weight applied as total evaluation. Cost shall be put heaviest weigh since it is the most important factor.

(2) Estimated cost of selected bridge is the base cost which will be used in the economic analysis. The procedure is described in next Clause 2.3.8 Economic Evaluation. It is basically the same for all the bridge in any terrain.

2.3.8 Economic Evaluation

2.3.8.1 General Principals

(1) General principals of economic evaluation of bridge implementation projects are basically the same in any terrain or location. It significates that no matter where is the bridges site is it shall be evaluated in same manner. This secures the methodology of comparing the various projects and selects more valuable projects to be implemented in order.

(2) Bridges being a part of highway/expressway, the economic value of the bridge implementation is included in that of highway/expressway project. Therefore, when the main object to be implemented is highway/expressway and bridges form parts of it, economic evaluation for implementing only bridges are not necessary and the project is valued by the economic evaluation of highway/expressway project. However, if the projects are mainly for implementing bridges, economic evaluation for bridge implementation project shall be conducted. Generally, bridge implementation is economically evaluated in major bridge implementation. Bridge implementation is evaluated economically in the project which implement new route of river crossing. The bridge does not have to be major bridge in this case since the majority or the project cost is bridge construction. However, economic evaluation shall be conducted for the entire project. Not only the bridge construction but also approach road, river training works and also other items shall be included. This principal is described in Clause 5.8.1, IRC SP:54-2018, and also in IRC SP:30-2009 'Manual on Economic Evaluation of Highway Projects in India (Second Revision)' (hereinafter called IRC SP:30-2009).

2.3.8.2 Period of Analysis

(1) Period of analysis is set from 15 to 30 years in general, as is described in Clause 5.8.4, IRC SP:54-2018. This is the same for the bridge, highway/expressway projects in any terrain.

2.3.8.3 Traffic Projection

(1) Traffic projection shall be conducted based on the traffic survey result at the site, mentioned in Clause 2.3.6.4. It could be done in the adjacent area if the traffic network analysis is possible. However, when the road network is poor, there is much less existing traffic volume compared to the needs of regional economic condition, such as both side of valley separated by river. Traffic survey result underestimates the existing traffic condition of the region. This situation can be occurred easily in hilly area and it shall be carefully adjusted.

(2) The bridge connects separated economic zones, there would be more generated traffic than bridges in different situation. Bridges in hilly area usually generates new traffic by its implementation much more than the bridge in other region. Generated traffic shall be carefully adjusted in some bridge projects in hilly area.

(3) Not only the growth rate of GRDP or regional economy related index, but also vehicle related index shall be taken into account in setting growth ratio of traffic volume in traffic projection, such as car ownership ratio, registration number of vehicles, etc. Bridges in hilly area tends to be located in rural area. Vehicle is not widely spread to local people yet, but it will spread to people more abruptly than in the urbanized regions.

2.3.8.4 Project Cost and Scheduling

(1) Project cost which is calculated in Clause 2.3.7.13 and selected in Clause 2.3.7.16 requires to be separated to annual cost by construction schedule prepared in Cause 2.3.7.11.

(2) Tax and subsidies shall be excluded for economic evaluation of bridge projects in hilly area as same as in other terrain.

(3) After the opening, operation, maintenance, and rehabilitation cost shall be taken into account annually. Operation cost is usually the organizational cost for managing department. Maintenance cost consists of the costs for routine inspection, periodic inspection, cleaning of road surface/drainage pipes, weeding, routine repair, snow cleaning, etc. Rehabilitation cost consists of the cost for repainting of minor damages, overlaying asphalt pavement, replacement of expansion joint/bearing, etc. maintenance and rehabilitation schedule shall be made in details, but it can be summarized as average annual cost for maintenance cost and concentrated to certain period, i.e. once in ten years, for rehabilitation cost.

(4) Operation, maintenance, and rehabilitation cost tends to be larger than the bridges in other terrains since the mountain bridges needs to face the severer natural condition. Therefore this tendency shall be taken into account in calculation of average cost for it.

2.3.8.5 Project Benefit

(1) Project benefit can basically be calculated in the same manner in any terrain. However, saving amount of time tends to be bigger in hilly area due to its lack of highway/expressway in comfortable alignment or river crossing points. Therefore, this tendency shall be considered in calculation of project benefit in bridge project in hilly area.

(2) Details of calculation of project benefit is described in IRC SP:30-2009

2.3.8.6 Sensitivity Analysis

(1) Sensitivity analysis shall be conducted by same manner for the bridge projects in any terrain, as mentioned in IRC SP:30-2009.

2.3.9 Feasibility Study Report

2.3.9.1 Preparation process of Feasibility study report is described in Clause 5.9 in IRC SP:54-2018.

2.3.9.2 It shall contain the unique condition of the bridge site in hilly area for justification of cost increase. Most of the project in hilly terrain requires higher cost than the projects in plain terrain. The report is the evidence to justify this cost increase and key for successful project implementation.

2.3.10 Acceptance of Feasibility Study Report

2.3.10.1 When the Feasibility study report is submitted, the project owner shall decide whether to implement the project or not. The report is the evidence for the decision. Therefore, the report shall be easily understood to the decision makers who are not familiar to the bridge site or the region, nor bridge engineering. This task is quite difficult task to be accomplished, but the effort shall be put into force.

2.3.10.2 Project owner shall understand and keep in mind that the mountain bridges has severer natural condition which is the cause of cost increase compared to the bridge in other terrains. This severe natural condition derives the project vulnerable and the cost and schedule could be effected negatively during the implementation of the project. Most of mountain bridges tends have this characteristics and it shall be taken into account in deciding the acceptance of Feasibility Study Report.

2.3.10.3 Further stage of the project is detailed designing, preparation of 'Detailed Project Report', followed by preparation of bid document, selection of contractor.

2.3.11 Selection of Design Consultant (added)

2.3.11.1 After the Feasibility Study Report is accepted and approved by the project owner, and the project owner decided to proceed to execute further stages for project implementation, there shall be the selection of design consultant.

2.3.11.2 Procedure of the design consultant depends on the project implementation scheme. If the project is PPP/DB projects, the selection of consultant will be included in the selection of the contractor. Recently in India, the projects of major bridges, highways/expressways are mostly implemented whether PPP or EPC scheme even for hilly area. Therefore, the next stage should be selection of the contactor in this case. However, to select the contractor, Detailed Project Report (hereinafter called DPR) shall be prepared, accepted, and approved by the project owner. In this case, 'Chapter 3 Design' shall be referred to for preparing the DPR.

2.3.11.3 For projects for minor bridges, detailed design shall be done and DPR shall be prepared by an experienced consulting firm for bridge designing. Bidding process shall be done according to the regulation of project owner or relevant authority. For DPR preparation, 'Chapter 3 Design' shall be referred to. The process in this clause 2.3.11.2 for bridges in hilly area is as same as for the bridges in other terrain.

2.3.11.4 Since the site of the mountain bridge is unique and fragile to natural disaster, the natural condition of the site changes considerably during the construction phase. It will vary much more in operation stage. Therefore, the Design Consultant shall be aware of this and have sufficient experience of coping this issue.

2.4 Preparation of Next Step in Bridge Implementation Project

2.4.1 Contract Document

2.4.1.1 Contract document for DPR consultant shall include all the necessary and important findings and results of analysis which are revealed till the feasibility study stage. Specifically the facts that is unique to the site and affects the work volume shall be carefully considered.

2.4.1.2 Special field and laboratory survey which shall be executed additionally to feasibility study shall be specified, and the quantity and the specification shall be prepared in details. This could be executed by well experienced engineer who has the knowledge of designing mountain bridges and information of the site. Project owner shall procure a well experienced engineer who knows about the site apart from design consultant for advisory service.

2.4.2 Selection of design consultant and DPR consultant

2.4.2.1 The detailed design of mountain bridges shall be execute by well experienced project manager, bridge engineer, team of other necessary engineer, shall be managed by well experienced consulting firm. Specification of required experience of project manager, different engineers and consulting firm shall be prepared in details. The experience of designing same type of superstructure in hilly terrain is one of the most important experience for the important personnel of the project.

2.4.2.2 Feasibility Study Report and other analysis reports shall be opened to the candidate consultant who intends to apply for the selection. Specifically the subsoil analysis report, hydrological analysis report, topographic survey report shall be opened so that the candidate consultant could prepare high quality proposal.

2.4.2.3 Design of the bridge would define the cost of the project and also the efficiency of the maintenance. Repair cost of the bridge are generally expensive and it is even more expensive in long span bridges. If the project includes the long span bridges, preventive structure or materials shall be given the advantage in proposal to decrease the life cycle cost.

2.4.2.4 Construction plan could also give big impact to the project cost and surrounding environment. Therefore, the construction plan which minimizes the cost as well as negative impact to natural environment shall be given advantages is selecting the winner.

CHAPTER 3 DESIGN

3.1 General

3.1.1 Since National Highways Development Project has been started, Indian government developed and applied many new procurement schemes for highway implementation, such as PPP, VGA, EPC, HAM, and etc. These schemes include detailed design stage together with construction, operation & maintenance stage except EPC, in the contract which is different to the conventional procurement process of government in highway implementation projects that is the engineering consultants prepare the detailed design report and the construction contractors perform the actual construction. This conventional process requires two individual contacts between the governmental implementation agency and consultant/contractor. The schemes do not require separated contracts and two contracts are usually merged to single contract.

3.1.2 However, for decision making of project implementation for governmental financing/implementation organizations, Detailed Project Report which was conventionally prepared to execute detailed design and to select construction contractor is still necessary. Nowadays, detailed design is done by new scheme contractor just before the commencement of construction taking advantage of their experience and technology. Therefore there are two steps for preparing detailed design for the construction, one by DPR consultant and another is by PPP/EPSContractor.

3.1.3 Taking into consideration of the above mentioned situation in India, this guideline treats technical aspects of design which shall be done in before the commencement of construction. It does not mention the exact timing the contents shall be executed. The related bridge engineer who belong to both DPR consultant and PPP/EPSContractor shall pay attention that all the necessary contents covered by this guideline could be explained in the report/drawings which they submit to governmental implementation agency.

3.1.4 This guideline concentrate on design of mountain bridges in both mountainous terrain and steep terrain (refer to IRC SP 73-2015) which has its length longer than 6m.

3.1.5 This guideline aims to supply additional but necessary information of designing mountain bridges which are not mentioned in already issued IRC Codes for realising safe/efficient/economical implementation, long service life span, lower life cycle cost, resilient to natural disasters. It shall contribute for the regional development in mountainous region in India.

3.1.6 Basic contents of bridge design is already described in various related IRC Codes. This guideline aims to complement the contents in design which are considered as necessary based on Japan's experience in implementing Mountain Bridges in Japan in last half century including advanced technologies. It is expected to contribute in improving the quality of implementation of Mountain Bridges in India, and accelerate the implementation period in implementing the highway network in mountainous regions.

3.1.7 Subchapter 3.2 treats the contents related to the detailed design for preparing "Detailed Project Report" which concentrate the mountain bridges. This Subchapter supplements the Chapter 6, IRC SP:54-2018. The contents are added based on the experience of Japanese construction industry, which has been constructing roads and bridge in the mountain and hill terrain in Japan. Japan has more than 70% of share of mountain and hill terrain geographically. Several state of art technologies is presented for the reference.

3.1.8 Subchapter 3.3 explains Quality Assurance System in terms of quality control items which shall be added to the quality assurance system of flat terrain.

3.1.9 Subchapter 3.4 explains the necessity of stating the preparation of the Maintenance Manual from the design stage.

3.1.10 Subchapter 3.5 describe seismic design methodology based on Japanese 'Specification for Highway Bridges' to realize high resilient bridge against strong earthquakes expected to occur in Himalayan Regions. Since Indian Code for bridge obligates limit state design and static analysis are basic methodology, there is not any description about dynamic analysis of seismic design. Japanese bridges have suffered with damage by earthquake from the beginning of its modernization. It came up with a conclusion that bridges with complicated structure, such as long span, high pier, integrated structure of substructure and superstructure, statically indeterminate structures of high orders, would be too massive if they are designed by static analysis and dynamic analysis contribute to implement with efficiency and strength, that is resistance capacity to earthquake seismic force. This subchapter introduce basic policy and methodology of seismic design of Japan focusing the dynamic analysis.

3.2 Detailed Project Report

3.2.1 General

3.2.1.1 Detailed Engineering stage is in project preparation is defined as the third stage of the preparation and described in Chapter 6 of IRC SP:54-2018 as 'PREPARATION OF DETAILED PROJECT REPORT'. In this stage, detailed surveys and investigations are carried out on approved site alignment. Detailed designs are worked out on the basis of results of survey and investigation and detailed drawings are prepared. An accurate cost estimate is made along with the Bill of Quantities and the specifications are finalised.

3.2.1.2 The detailed project report (DPR) incorporating all above mentioned aspects is prepared. DPR should clearly indicate the required budget provision for the project, implementation schedule and also include documents for pre-qualification of contractors and the tender documents when required. In addition, the quality assurance system and maintenance manual shall be prepared for construction and operation stage.

3.2.1.3 Detailed Project Report is the very basic document of decision making of actual project implementation since after this there will be construction contractor assigned to the project. Proper attention shall be paid for project implementation cost since insufficient estimate could end up in shortage of budget during the construction stage. The mountainous terrain could be very different in geological condition by locations and rough survey, design, and analysis/study are likely to be a cause of enormous additional budget for completing the project. Even the construction were completed, lack of budget could lead to low quality or durability of the bridge and its surrounding facilities. This issue would increase life cycle cost of the project and give negative impact to the regional economy. This stage in the mountainous terrain needs closer attention than that in the plain terrain.

3.2.1.4 This subchapter's structure follows the structure of the Chapter 6, IRC SP:54-2018.

3.2.2 Detailed Investigation

3.2.2.1 Surveys/studies to be executed (added)

(1) Detailed investigation is required to finalize the location of bridge site, alignment of bridge. In mountain and steep terrain, a few meters of alignment shift could end up with several 10 meters of bridges length increase. Therefore, following surveys and investigations shall be executed to prevent unexpected increase of construction cost after the commencement of the actual implementation;

Table 3-1: Required Detailed Investigation to Finalize the Location of Bridge Site

- topographic survey
- hydrological survey, (not only discharge, but also debris flow)
- subsoil investigations (not only for foundation, but also land/mud slide)
- environmental impact assessment (endangered species of fauna and flora, habitat of minority, etc.)
- procurement survey (unit cost for materials, transportation cost, etc.)
- site survey for construction planning

(2) For investigating and clarify the effect of wind to long span bridge, such as suspension bridge, cable stayed bridge, and other types of superstructure with its longest span length exceeding 200m, the needs of Model study shall be studied considering the effect of surrounding terrain. The direction and the speed of local wind/gust can only be analysed by model study. The phenomenon in hilly area is much more complicated than that of flat terrain and it is very difficult to predict just by the analysis on the desk. It shall be carried out immediately after acceptance of approach alignment and bridge site. Once the basic data is collected by model study, it can be utilized and applied to digital model simulation analysis for more detailed analysis.

3.2.2.2 Final Location Survey

(1) The main purpose of the final location survey is to fix the centre line alignment. This is the same purpose for mountain bridges with that of bridges in flat terrain. The engineers shall keep the fact in their mind that several meter shift of alignment could raise the bridge length by several 10 meters at the site with very steep slope. However, final location of the bridge is proposed and will be approved by the result of Detailed Project Report Study. Therefore it shall cover sufficiently wide area to be ready for necessary adjustment.

(2) Since the terrain in hilly area is complicated, actual filed survey by checking topographic survey results by responsible bridge engineer shall be done for final checking of site terrain.

(3) Updating and upgrading to topographic survey data is recommended by preparing as fine as 1:100 scale of topographic map specifically the river section and substructure locations.

(4) Actual field survey by responsible geologist shall be executed to identify the possible location of landslide. Location of substructures shall avoid the influence area of landslide or areas prone to landslide disaster since landslide would displace the substructure massive amount and trigger superstructure fall. It leads to fatal collapse of the entire bridge system when the superstructure is connected as multi-span structure. Traffic would be closed for months even some parts remains survived even for span-by-span superstructure bridges.

(5) Actual field survey including the actual site survey of upstream region of catchment area of the river flow shall be executed by responsible hydrologist to clarify the risk of the debris flow. Substructure shall not be placed within river flow where the site location is facing high risk of debris flow. Debris flow would also displace the substructure and lead to fatal destruction of the entire bridge system very similar to by the effect of landslide which is mentioned in previous clause.

(6) Actual field survey by responsible road engineer shall be carried out when the bridge is planned to be widened or replaced by new bridge for planning the detour for existing traffic. The slope could be very fragile for securing extra area for detour by cutting or filling. Temporary structure(s) or/and slope protection work(s) are recommended to be planned for protecting slope at site as well as natural environment of surrounding area.

(7) There are another purpose of the actual field survey by responsible road engineer when the bridge/highway project is new implementation. It is to secure the temporary access road specifically for foundation construction for piers. The length of the temporary road would be considerably long when the slope is steep and valley is deep at the construction site. It could raise the construction cost and also harm the natural environment of the surrounding area. Therefore, collecting the necessary information for designing the temporary access road by responsible road engineer is very important to successful implementation.

(8) For construction planning and cost estimates, procurement survey and interviews shall be executed to secure the quality of both construction plan and required construction costs. There are no IRC Codes for these for these items, and the surveyors shall collects the data/information of availability of the facilities, equipment, materials, and etc. Departments of states which are in charge with road implementation, operation and maintenance usually has the standard unit prices for common facilities, equipment, materials, of procurement. However, when the bridge site is far from state capital, the value of unit price could differ considerably. The surveyors (actual data collectors) and engineers/specialists (managers) shall keep this in mind and avoid being reluctant for excusing actual field survey and using the standard unit prices without checking the actual value differences.

3.2.2.3 Sub-soil Investigation

(1) Sub-soil investigation by boring survey shall be at the location of every substructure including abutments and piers to define the depth of bearing soil layer and clarify other characteristics geotechnically.

(2) Sub-soil strata tends to be inclined in hilly area specifically in the steep terrain. It is not rare that bearing layer has more inclination than surface slope. Therefore, more than 2 boring survey shall be done to specify the angle of the layer. To specify the direction of the inclination, at least 3 boring survey is required. This type of survey shall be carried out in steep angle and large spread foundation or large number of pile foundation.

(3) The result of boring survey shall be utilized to plan the foundation construction method. The size of boulder, particle size distribution, and ground water level are the most important data for the selecting foundation type, designing the foundation, and planning the foundation construction method. Detailed correspondence between boulder size and types of foundation/construction method is demonstrated in 3.2.3 and 4.4.

(4) The result of boring survey is one of the reliable evidence of land/mud slide. If remarkable traces or signs of slide is found in this stage, the substructure location shall be reviewed or other countermeasure to resist against the slide shall be planned.

(5) Details of sub-soil investigation is defined and explained in Appendix-2, IRC 78-2014. Mountain bridges shall also follow this specification for detailed sub-soil investigation for detailed designing.

3.2.2.4 Environmental Impact Assessment

(1) Necessity and requirements of Environmental Impact Assessment shall be referred to the related guidelines and specification issued by Ministry of Environmental and Forest and Climate Change (MoEFCC), India.

(2) It is very often that the sites of bridge construction in mountainous and steep terrain and its surrounding areas are very prone to be harmed or destructed by construction procedure. The points to be focused in the assessment in bridge implementation projects in hilly area is mentioned in Clause 2.3.7.14 Initial Environmental Examination. Special care and attention shall be payed when there exist rare or endangered species both of fauna and flora.

3.2.2.5 Social Impact Assessment, Land Acquisition and Resettlement Action Plan

(1) Apart from natural environment, there shall be social environmental impact shall be assessed considering the small numbers of population which is impacted by bridge construction in hilly area. Special care and attention shall be given when there exists ethnic minority and site of construction is in their daily living area.

(2) Confirmation shall be executed that the bridge alignment and its ROW does not effects public facilities, such as hospital, school, temples, church, historical monument, and etc.

3.2.2.6 Summary of Detailed Investigation (Added)

(1) Above mentioned investigations/surveys results shall be summarized so that it could be utilized and applied to detailed design as the design conditions. The contents of design conditions differ by the objects which are to be designed. There are design conditions correspondence to different parts for bridge, such as the entire bridge, superstructure, substructure, foundation, and each members such as girders, slab, column, pile, and also other miscellaneous parts. Some conditions are common for most of the objects and some are specific to objects.

(2) The details shall be referred to relevant specifications of IRC. Followings are the main IRC Codes

Table 3-2: Main Relevant Specifications of IRC to be referred for Detailed Investigation

IRC 5 2015	SSCPRB, Sec. I (General Features of Design) (8th Rev.)
IRC 6 2014	SSCPRB, Sec. II Loads and Stresses (4th Rev.)
IRC 22 2015	SSCPRB, Sec. VI - Composite Construction (Limit States Design) (3rd Rev.)
IRC 24 2010	SSCPRB, Sec. V Steel Road Bridges (Limit State Method) (3rd Rev Rev.)
IRC 40 2002	SSCPRB, Sec. IV (Brick, Stone and Cement Concrete Block Masonry) (2nd Rev.)
IRC 78 2014	SSCPRB, Sec. VII, Foundations and Substructure (Revised Rev.)
IRC 83-1 2015	SSCPRB, Sec. IX (Bearings), Part I (Metallic Bearings) (2nd Rev.)
IRC 83-2 2015	SSCPRB, Sec. IX (Bearings), Part II (Elastomeric Bearings) (1st Rev.)
IRC 83-3 2002	SSCPRB, Sec. IX (Bearings), Part III (Pot, Pot-Cum-PTEE, Pin and Metallic Guide Bearings)
IRC 83-4 2014	SSCPRB, Sec. IX (Bearings), Part IV (Spherical and Cylindrical)
IRC 112 2011	Code of Practice for Concrete Road Bridges

Note) SSCPRB: Standard Specifications and Code of Practice for Road Bridges
Sec.: Section, Rev.: Revision

(3) IRC Codes are not divided by structure, more members apart from foundation and substructure: IRC 78 and bearings: IRC 83. It is divided by materials, such as concrete, steel, composite, brick, stone, cement concrete block masonry, and etc. Therefore, the responsible bridge engineers shall understand the structure and objectives of the chapters and clauses of these IRC Codes and extract the required conditions. The results of the detailed investigation shall be summarized for bridges engineers to set the design conditions. Therefore required data shall be collected and summarized as the investigation/survey report.

(4) Investigation/survey of and surroundings of the site in the mountainous and steep terrain, has wide variety of results which shall be summarized for detailed designing. Collecting inappropriate data/information or analysing collected data/information unsuitably could leave incorrect estimates of procurement of materials, equipment, and facilities as well as the implementation budget. It could end up the project with increase of enormous budget and moreover to bankrupt in extreme cases.

(5) The bridges which are to be implemented in these terrain are much more prone to face natural disasters which could lead the bridge to collapse or leave severe damages. Collecting inappropriate data/information or analysing collected data/information unsuitably could leave fatal damages to the bridge during the operation and increase the life cycle cost, decrease operational life span, leave hazard to the traffic, and give negative impact to regional economy along the route.

3.2.3 Detailed Design

3.2.3.1 General

(1) After the investigation/survey results are summarized and handed to responsible bridge engineer, the engineer has to prepare the design conditions based on the design philosophy developed in the beginning stage of the bridge preparation. These conditions are utilized in the structural calculation following the methodologies defined in the IRC Codes for bridge design.

(2) The purpose of detailed design is to prepare the detailed engineering drawings not just for the Contractor to prepare the shop/working drawing for construction works, but also for calculating quantity, estimating cost, preparing necessary data/information of materials, equipment, and facilities.

(3) And also the bases and the conditions prepared for calculation, necessary specifications for construction are prepared to secure the quality of the bridge after the implementation. With all of the documents/drawings/specifications prepared in the detailed design, preparation for actual construction works can be commenced.

(4) Mountain bridges have unique conditions which are dependent upon the terrain, region, climate, sub-surface strata, and even orogenic activity. Therefore, collected data/information and experience of site reconnaissance shall be given the priority more than the results of desk/book analysis in setting the design conditions for the bridges in mountainous and steep terrain.

3.2.3.2 Procedure (Added)

(1) The procedure of detail design for implementing mountain bridges from fixing the bridge alignment is as follows;

Table 3-3: Procedure of Detail Design for Implementing Mountain Bridges

1) Fixing the bridge alignment
2) Fixing the bridge length: comparison study of abutment (location, type, height)
3) Fixing the span arrangement:
    i) Extracting location where constructing substructure is not feasible
    ii) Setting the alternatives of span arrangement
    iii) Applying the superstructure type to the alternatives of span arrangement
    iv) Structure calculation and cost estimates for each alternatives of iii) and select around 3 alternatives [preliminary design].
    v) Structure calculation, cost estimates and construction planning for 3 alternatives of iv) and select the most appropriate span arrangement and superstructure type [basic design].
4) Structural calculation for superstructure, substructure, foundation, bearings.
5) Seismic analysis for reviewing the results of 4).
6) Detailed design for other miscellaneous members.
7) Preparing detailed drawings and construction planning/schedule.
8) Quantity calculation and cost estimates.
9) Review from environmental point of view.
10) Preparing the necessary specifications based on the result of detailed design.
11) Preparing the Detailed Project Report.

(2) Further stages are followed by the items as follows,

Table 3-4: Further Stages of Implementing Mountain Bridges after Detaied Design

1) Preparing the bidding documents for selection of the Contractor.
2) Executing the bid and selecting the Contractor.
3) Reviewing the detailed design result and prepare the shop/working drawings
4) Obtain approval of the shop/working drawings from relevant authority.
5) Obtain approval of materials, equipment, facilities from relevant authority.
6) Actual construction works execution with environmental countermeasures if required.
7) Prepare as-built drawings and be inspected for completion.

(3) There is not any special process in designing the mountain bridges. The approach and methods are different or some extra contents are required in some steps. Especially the structure types which have advantages for mountain bridges, such as high RCC piers, balanced cantilever rigid frame box-girder, arch bridge, suspension bridge, etc. The details are described hereinbelow.

3.2.3.3 Design Conditions (Added)

(1) Geometry

1) Since this guideline treats highway/expressway bridges, geometry of the bridge is designed based on the specification and codes of highway/expressway. These condition are demonstrated as cross section (carriage way, shoulder, median, kerb, footpath, railing, protect fence, pavement, and clearance), cross fall, superelevation, plan alignment (straight, arc, clothoid curve), longitudinal gradient (slope, sag, crown), change of cross section (merging/dividing lane, climbing lane for slower traffic, securing safety sight distance, etc.), others (emergency parking bay, toll gate facility, and etc.,).

2) These highway/expressway alignment related standard/specification are mainly described in IRC:52-2001 for Hill Roads, IRC:73-1980 for Rural Highways. The details are set by responsible highway engineer and the conditions for bridge design are provided accordingly. It is also summarized in IRC:5-2015 for any types of bridges.

(2) Natural Conditions

1) Topography: topographic data/information shall be collected and analysed based on the actual site survey and 3-dimentinal data from the survey shall be demonstrated on 3-dimentional CAD drawings. Scale and accuracy of the data/information shall be sufficient to print out in 1:100 with contour line and discontinuity location, such as rocks, cliff, and hollow, artificial structure, and etc.

2) River, Hydrology: River and hydrology design conditions, mainly the discharge, shall be defined from the analysis of actual site, catchment area, and upstream fiver flow of the crossing river. The contents which shall be defined are summarized in IRC:5-2015. (There is IRC SP:19-2001, however the description is not very in details)
If bridge site is prone to debris flow, special consideration shall be done for the location of piers. If the quantity of the debris or size of rocks included in the flow has fatal impact to pier(s), any piers shall be placed in the river cross section. If any of the location of piers cannot be avoided within the river flow, sufficient protection structure shall be installed to protect the pier to prevent damage from debris flow. The protection structure is useful when the quantity of the expected debris is small. The results of analysis are the basis of calculation the force and load in detailed designing and it shall be prepared by the responsible hydrological engineer. Special attention shall be given due to the possibility that is could the critical case for designing pier and its foundation. Span arrangement shall be reviewed if there are new findings of debris flow after the feasibility study.
There are also horizontal force by river flow. It is described in IRC:6-2014. However, it is based on the river flow velocity and not the density of the flow, when the piers and protection structure are located in the debris flow, the influence of the debris shall be taken into account applying academic methodology.

3) Soil, Geotechnics: Soil and geotechnical condition shall be defined from the analysis of actual survey and laboratory tests. The contents which shall be defined are summarized in IRC:5-2015. (There is IRC SP:19-2001, however the description is not very in details)
If bridge site is prone to land slide, special consideration shall be done for the location of piers. Any pier shall not be placed where the slope has high possibility land slide. It can be located when sufficient countermeasure is installed for the slope and the risk can be neglected. Span arrangement shall be reviewed if there are new findings of land slide after the feasibility study.
Sub-soil give load and resistance to the displacement of bridge structures. The required data/information to define these design conditions are summarized in IRC:6-2014.

4) Wind: Wind velocity shall be collected by the actual record in the observatory nearby. If there is not any reliable data, academic methodology to assume the velocity shall be applied. There shall be several methodology compared in previous stages, specifically in the feasibility study. However there shall be a review to define the condition which give stronger load for safety and stability of the bridge. The detailed description is shown in IRC:6-2014 for uniform wind direction.
Wind does not blow only in horizontal direction at some valley or along the slope in mountainous region. There for the condition shall be confirmed by the model study if there is upward/downward strong wind.

5) Earthquake: Seismic condition is summarized in IRC:6-2014. If there are larger earthquake occurred recently than the data for IRC:6-2014, There shall be a review and the condition has to be revised. The historical record has been broken recently, in these several decades, worldwide. Blind trust to IRC Code can lead a fatal collapse by earthquake in the Himalayan Mountains region. The senior bridge engineer who is responsible for design result shall take special consideration with advice by responsible geologist.

6) Climate: Temperature range shall be set as the design condition not only for temperature load but also for the selecting appropriate materials. There is description in IRC:6-2014 for temperature range by the region. However, if the bridge is located in high altitude region, the influence of altitude to the temperature shall be taken into account.
Snow cover is a type of dead load. The load is not depend of the depth, it is dependent to liquid water content of snow. Himalayan mountains region are snow zone and needs to consider the snow load. IRC:6-2014 has its description but it does not define the amount/value by region. Actual record of snow cover depth shall be collected and analyse based on the return period set for the bridge.

7) Others: Some chemical materials impacts shall be considered if the bridge is located near the affected area. Actual quantity of exposure shall be collected. If the expose amount differs by season, the maximum amount shall be collected. If the reliable record is not available, it shall be assumed by applying the academic methodology. Assumption shall give the results which give stronger load for safety and stability of the bridge. This condition is applied to select the materials, protection methods, and maintenance specification.

(3) Traffic Condition

1) Traffic volume data is collected for defining the typical cross section of highway/expressway including the bridge. It is defined by the results of traffic demand forecast. The design condition is given as the typical cross section and it is applied to the plan of the bridge. If the bridge has change of cross section (merging/dividing lane, climbing lane for slower traffic, securing safety sight distance, etc.), others (emergency parking bay, toll gate facility, and etc.), the width of the bridge will change accordingly. These conditions are given by responsible highway engineer's decision. Among these changes, climbing lane for slower traffic, securing safety sight distance are the most related changes in highway/expressway in mountainous region. These conditions are normally defined in previous stage and not necessary for review nor revise. However, bridge engineers shall pay attention for the modification in highway design.

2) Highway/express way crosses over the existing/planned district/local roads and railways which need vertical and horizontal clearance for the traffic to run under the bridge safely. These conditions are normally defined in previous stage and not necessary for review nor revise. However, bridge engineers shall pay attention for the modification of crossing road/railways, specifically if they have improvement plan.

(4) Loads

1) Load conditions for bridge designing is summarized and details are shown in IRC:6-2014. Types of loads described in IRC:6-2014 are Dead Load, Live Load, Snow Load, Impact due to floating bodies or vessels, Vehicle collision load, Wind load, Water current, Longitudinal forces caused by tractive effort of vehicles or by braking of vehicles and/or those caused by restraint of movement of free bearings by friction or deformation, Centrifugal force, Buoyancy, Earth pressure including live load, surcharge, Temperature effects, Deformation effects, Secondary effects, Erection effects, Seismic force, Wave pressure, Grade effect.

2) The types load which are required close attention in designing bridges in hilly area are Snow Load, Earth Pressure, Temperature effects, Seismic force. Design conditions to define these types of load shall be closely defined by reviewing the conditions set in previous stages by adding the new obtained/collected data/information.

(5) Materials

1) Specification of materials are defined in different IRC Codes as follows,
Concrete related materials (Cement, Aggregate, re-bars, admixtures, etc.) in IRC:112-2011
Steel materials (Steel bridge materials) in IRC:24-2010
Soil materials (brick, stone, block masonry, fill) in IRC:40-2001
Bearing materials (steel, elastic rubber, etc.) in IRC:83 1 to 4
River protection materials in IRC:89-1997

2) Other materials which are required in specific types of structure are defined in relevant IRC Codes, for instance materials used specifically for expansion joints are defined in IRC SP:69-2011, and materials applied in high performance Concrete is defined in IRC SP:70-2016. There are more IRC Codes with SP which has specific materials defined for specific use in the structure.

3) There are no specific materials specifically applied for mountain bridges. High piers require high performance concrete, long span steel bridges require high strength high tensile strength steel and more materials which are not applied to ordinary small bridges are also applied for mountain bridges. However, these materials are not used just for mountain bridges and used for bridges in other terrain.

3.2.3.4 Superstructure

(1) General

1) Basic methodology to design superstructure is basically same for all types of superstructure. However, the indeterminate superstructures requires more complicated calculation and analysis than that of determinate superstructures. Therefore, single span superstructures are most simple for designing and with increase of degree of redundancy of superstructure (bridge itself when superstructure and substructure are connected rigidly), calculation and analysis gets more complicated and requires software application to calculate. Rigid frame structure, multi-span continuous superstructures, and cable stayed bridges are the most complicated structure for calculation and analysis.

2) The difference of main materials, such as cement concrete or steel, requires different methodology and details, such as shape and allocation, of each member are separately defined. Therefore, process and details of detailed design of superstructure shall be defined by its structure.

3) For complicated structures and/or massive structures (long span bridges and bridges with long continuous superstructure) or superstructure sustained by multi-piers with high piers or considerably different height requires dynamic analysis especially for seismic analysis, and analysis to clarify the effect of wind. Necessity of the model analysis shall be studied and executed when required.

4) Mountain bridges of highway/expressway tends to have high piers, and different height of piers with long span length. Their superstructures often connected to substructures rigidly. Moreover, Himalayan Mountains regions are with high risk of strong earthquakes and the wind velocity is strong. Therefore, the possibility of necessity of dynamic analysis and model study is higher than bridges in other regions.

5) Construction restraints are much severer in hilly area than other areas. Therefore, span arrangement are influenced by construction condition of substructure/foundation. Span arrangement is usually studied and compared for selecting the most appropriate arrangement in previous stage, feasibility study. However, detailed design stage is the last opportunity for revision/modification of span arrangement. Review of span arrangement and construction plan shall be done before commencement of the actual designing.

(2) Concrete Superstructure

1) RCC Superstructure

i) RCC superstructures are usually applied for short span bridges shorter than 20m and in the regions where pre-tensioned PSC girders are not available. However, it shall not applied for heavy traffic highway/expressway routes due to rapid progress of fatigue of the reinforcement bars. It decreased service life and increase the Life Cycle Cost.

ii) Steel girder bridges can be applied for short span bridges shorter than 20m as well. However, transportation from the steel plant, girder manufacturing factory could cost very much in some regions. Not only the availability and difficulty of transportation, cost comparison shall be compared with RCC superstructures.

iii) If the number of short span bridge is economically sufficient for production of pre-stress girders, it shall be the best solution to minimize the maintenance cost as well as Life Cycle Cost.

iv) Service life of the RCC superstructure depends on the quality of construction. When the quality of construction is poor, deterioration of RCC itself will rapidly be accelerated. Therefore, when applying RCC superstructure, Close attention shall be payed to maintain the quality control level of construction high throughout the construction phase. Monitoring system of the quality assurance system shall be studied in detailed design stage.

v) Most of the members are able to be designed by IRC:112-2011 with the instruction described in IRC SP 105-2015. If the bridge is a small bridge (total length of 30m or less), IRC SP:13-2004 also shall be referred to.

vi) If the superstructure is cast-in-situ voided slab, IRC SP:64-2016 shall be referred to.

vii) If the superstructure is continuous for multi-spans, IRC SP:66-2016 shall be referred to.

viii) If the superstructure applies high performance concrete, IRC SP:70-2016 shall be referred to.

ix) Construction yard is often very limited in the mountainous region. Batching plant is ideally located very close to the casting site, but it is not always possible considering the natural environment protection. Aggregates of nearby borrow pits shall be applied to save the construction cost. Setting the location of crushing plant is also limited in the aspect of natural environment protection. Construction planning has trade-off issue of protecting natural environment and saving cost. It shall be closely studied to optimise both factors.

2) PSC Superstructure

i) PSC superstructure has very wide range of span length. It is applied from 6m to 250m or even more (world longest span for PC superstructure is 530m in Panama, Atlantic Bridge, PSCable stayed bridge, as for Aug. 2018).

ii) There shall be comparison study economically and technically among RCC and steel superstructure for short span range with pre-tensioned superstructure. The points to be considered in comparison study is described in previous clause for reference.

iii) There shall be comparison study economically and technically between steel superstructure and post-tensioned superstructure for the span longer than 20m.

The alternative for comparison study depend upon the span length of the bridge. The possible alternatives are shown Table 2-9 in the previous Chapter.

iv) PSC structures are basically very easy to maintain if the quality of construction is sufficient. However, if the quality is poor, it could derive quick deterioration of the structure. If the prestress cable are deteriorated, it could cause fatal damage to the bridge. Close attention shall be payed to maintain the quality control level of construction high throughout the construction phase. Monitoring system of the quality assurance system shall be studied in detailed design stage.

v) Most of the members are able to be designed by IRC:112-2011 with the instruction described in IRC SP 105-2015. If the bridge is a small bridge (total length of 30m or less), IRC SP:13-2004 also shall be referred to.

vi) If the superstructure is cast-in-situ voided slab, IRC SP:64-2016 shall be referred to.

vii) If the superstructure is segmental structure, IRC SP:65-2005 shall be referred to.

viii) If the superstructure is continuous for multi-spans, IRC SP:66-2016 shall be referred to.

ix) If the superstructure has external unbonded prestressing tendons, IRC SP:67-2005 shall be referred to.

x) If the superstructure has external unbonded prestressing tendons, IRC SP:67-2005 shall be referred to.

xi) If the superstructure applies high performance concrete, IRC SP:70-2016 shall be referred to.

xii) If the superstructure applies pre-tensioned girders, IRC SP:71-2006 shall be referred to.

xiii) Construction yard is often very limited in the mountainous region. Casting yard of PSC pre-tensioned/post-tensioned girders require considerable area and it shall be located close to the bridge site. Batching plant is ideally located very close to the casting site. However, it is not always possible considering the natural environment protection. Aggregates of nearby borrow pits shall be applied to save the construction cost. Setting the location of crushing plant is also limited in the aspect of natural environment protection. Construction planning has trade-off issue of protecting natural environment and saving cost. It shall be closely studied to optimise both factors.

xiv) Moreover, construction and erection methods for PSC superstructure in hilly area shall be studied in details to minimize the construction cost and also to mitigate the natural environmental impact. Considering the limited space for temporal construction yards, the balanced cantilever method of constructing the superstructure of bridge with long span is one of the alternatives with advantages, specifically when the span length exceeds 75m.

(3) Steel Superstructure

1) General

i) Steel superstructure can be applied to the most of span length technically and physically (refer to Table 2-9 in the previous Chapter). However, it cannot be applied anywhere due to is high procurement cost.

ii) However, steel superstructure has advantage in specific occasions, such as easiness of erection, rapid construction, and light weight. Specifically for single span bridge which has more than 50m span length with restriction of installing erection facilities, steel superstructure shall have more advantage technically and also economically. Therefore, single span arch bridge over deep valley, wide river/canal with restriction of placing piers in the water flow, etc. are the examples.

- **iii)** Disadvantage of steel superstructure is that the material itself is always facing the risk of being corroded. To protect the materials from corrosion, it is coated by paints usually. However, repainting including removing old paint is necessary every 30 years. Therefore, the maintenance cost of steel superstructure is normally higher than concrete structure.
- **iv)** There are material option of 'Weathering/Weatherproof Steel' which only requires paints at girder ends reduces life cycle cost by reducing the quantity of painting drastically both in initial and maintenance cost. Condition of application is shown in the following clause, 3.2.3.4 (3) 4) 'Weathering/Weatherproof Steel'

2) Plain Steel

- **i)** Designing steel superstructure by plain steel is described in IRC:24-2010.
- **ii)** Preparing work drawings (including welding, bolt connection), IRC SP:104-2015 shall be referred to. Rivet connection shall be selected with low priority due to the difficulty of replacement when damaged.
- **iii)** Planning erection methods, IRC SP:104-2015 shall be referred to.
- **iv)** Arch bridges, suspension bridges require specific method for erection. Therefore experienced bridge engineer shall be assigned for planning the construction/erection.
- **v)** Mountainous regions tends to be far from steel plant and manufacturing factory and transportation cost tends to be very high. Close review and investigation shall be done in selecting steel superstructure since the rough cost estimation in the precious stage could be underestimated. Cost shall be reviewed before the detailed design with the result of preliminary/basic design and after the detailed design with the result of detailed design.

3) High Tensile Strength Steel

- **i)** High tensile strength steel is applied to long span steel bridges to reduce deal load of the superstructure as well as its total material cost. It is normally defined that the steel with more than 500N/mm^2 is high tensile strength steel. However, for bridge material, more than 500N/mm^2 shall be called high tensile strength steel.
- **ii)** High tensile strength steel has not only high tensile strength but also high toughness, weldability, bending workability, and corrosion resistance. It has economical advantage in long span which is longer than 100m in general. Comparison study of materials of tensile strength shall be executed in detailed design to identify the material.
- **iii)** The details of the detailed design is described in IRC:24-2010 which shall be referred to.
- **iv)** Bridges for highway/expressway tends to have long spans specifically in steep terrain. There shall be considerable amount of opportunities that high tensile strength steel be applied in near future.

4) Weathering/Weatherproof Steel

- **i)** Weathering/Weatherproof Steel is one of the technology to resist against corrosion of steel material. Specific elements which are effective to resist against corrosion, such as Copper, Chrome, and Nickel, is added to the plain steel. This steel forms 'stabilized rust' uniformly on the surface of the steel material and this 'stabilized rust' protects the interior portion. It does not require painting for the entire service life if the airborne salinity is small and stable.

ii) Weathering steel is applicable in the region with certain distance from ocean coast line. It is regulated in Specifications for Highway Bridges of Japan that it requires from 1 to 20 km by region in four categories. Mountainous region in India is not very near to ocean coast line, therefore, there shall be wide region that weathering steel could be applied to reduce life cycle cost of the bridge.

iii) There is not any specification, manuals, or guidelines for weathering steel in India yet.

iv) The followings are the description of weathering steel in Specifications for Highway Bridges of Japan (Part II Steel Bridges, P. 82, Chapter 5.2 'Rust and Corrosion Prevention').

Quoted from Specifications for Highway Bridges of Japan (Part II Steel Bridges)

Chapter 5.2 Rust and Corrosion Prevention

Weathering steel is a steel with an appropriate amount of alloying elements added to form a fine rust layer on the steel surface. This rust layer protects the steel surface and prevents the subsequent progress of rusting. Thus, the corrosion rate is lower than plain steel. In order to allow weathering steel to deliver specified performance, it is necessary to use it under the environmental conditions suitable for it. For example, if weathering steel is used with no paint applied in some locations, a uniform and fine rust layer may be difficult to form in bridges subject to the influence of chlorides or in environments that prevent the surface of the steel from undergoing appropriate wet-dry cycles. Such locations include those where weathering steel is often exposed to airborne salinity, those where an antifreezing agent is used, or those that are close to a location where an antifreezing agent is used. To ensure the durability of the entire bridge, it is also necessary to consider another type of corrosion protection, even in cases where weathering steel is used in painted bridges. For example, partial painting may be used in locations with poor local environments, such as the edges of girders, which are thought to require sufficient measures to be taken for corrosion protection.

For example, the weather-resistant hot rolled steel for welded structure prescribed in JIS G 3114 may generally be used in unpainted form3l in a region where the airborne salinity measured by a prescribed method does not exceed 0.05 mdd (NaCl: mg/100 cm^2/day) or in the region shown in Figure C. 5.2.1. In this case, the suitability of use of weathering steel in unpainted form must be evaluated on the basis of the measurement of airborne salinity. This evaluation requires sufficient understanding of various factors, such as the distance from the coastline, regional characteristics depending on meteorological conditions, seasonal changes, and annual changes. On the basis of this understanding, the suitability of use needs to be carefully evaluated by comparing the results of previous studies on the surrounding area of the cross-linking point. This suitability of use is evaluated from the perspective of the regional environment. It is also necessary to give separate consideration to each bridge with regard to its local corrosive environment; this differs depending on various factors, such as geological conditions at its cross-linking point and the structural detail of its members.

For rusting condition on weathering steel surface, at the initial inspection in service stage, there is a need to verify conformance to environmental condition of each bridge and to survey defects caused by local corrosive environment. When inspecting periodically, changes of environmental condition shall be considered, and, if abnormal rusting occurs, the causes thereof shall be eliminated and appropriate measures such as painting shall be taken as needed.

[Source: Specification for Highway Bridges of Japan (Part II Steel Bridges)]

v) From the description of Specifications for Highway Bridges of Japan (Part II Steel Bridges), the following conditions are defined for applying the weathering steel for Mountain Bridges.

Table 3-5: Conditions for Applying the Weathering Steel for Mountain Bridges

- The airborne salinity measured by a prescribed method does not exceed 0.05 mdd (NaCl: mg/100 cm²/day)
- It shall not be applied without protective paint for the bridges which an antifreezing agent is used, or those that are close to a location where an antifreezing agent is used.
- Partial painting shall be used in locations with poor local environments, such as the edges of girders (easily wetted by the rain leaked through the gaps of expansion joint), which are thought to require sufficient measures to be taken for corrosion protection.
- Suitability of use of weathering steel in unpainted form must be evaluated on the basis of the measurement of airborne salinity.

vi) Other characteristics of material are quoted from Specifications for Highway Bridges of Japan (Part II Steel Bridges).

Table 3-6: Steel Grade Selection Standard by Plate Thicknesses

Steel grade		Plate thickness
Steel for welded structure	SMA400AW SMA400BW SMA400CW	(6) 8 to 25mm (6) 8 to 40mm (6) 8 to 100mm
	SMA490AW SMA400BW SMA490CW	(6) 8 to 16mm (6) 8 to 40mm (6) 8 to 100mm
	SMA570W	(6) 8 to 100mm

Note: Steel materials less than 8 mm in plate thickness shall conform to Clauses 4.1.4 'Minimum Thickness of Steel' and 9.4.6 'Minimum Thickness of Longitudinal Rib'.

Quoted from Clause 1.6 'Steel Grade Selection'
(5): When using a steel material conforming to JIS G 3101 (Rolled steels for general structure), JIS G 3106 and JIS G 3114, its steel grade and through-thickness shall be selected based on Table 1.6.1 as standard (Table 3-1 in this guideline).

Table 3-7: Chemical Components of Weather-Resistant Hot-Rolled Steel for Welded Structure

Chemical Components / Steel Grade	C	Si	Mn	P	S	Cu	Cr	Ni	Others
SMA400W	0.18 or less	0.15 - 0.65	1.25 or less	0.035 or less	0.035 or less	0.30 - 0.50	0.45 - 0.75	0.05 - 0.30	For each steel grade, Mo, Nb, Ti, V, Zr and other elements that are effective for weather resistance may be added. However, the total amount of these elements shall not exceed 0.15%.
SMA490W	0.18 or less	0.15 - 0.65	1.40 or less	0.035 or less	0.035 or less	0.30 - 0.50	0.45 - 0.75	0.05 - 0.30	
SMA570W	0.18 or less	0.15 - 0.65	1.40 or less	0.035 or less	0.035 or less	0.30 - 0.50	0.45 - 0.75	0.05 - 0.30	

Table 3-8: Mechanical Properties of Weather-Resistant Hot-Rolled Steel for Welded Structure

Steel Grade	Tensile test								Impact test		
	Yield point or proof stress (N/mm^2)				Tensile strength (N/mm^2)	Elongation			Symbol	Test temperature** (°C)	Charpy absorption energy (J)
	Steel thickness (mm)										
	≤ 16	16 < and ≤40	40 < and ≤75	75 <		Steel thickness (mm)	Test piece*	Elongation (%)			
SMA 400W	245 or more	235 or more	215 or more	215 or more	400 - 540	≤16 16 < 40 <	No. 1A No. 1A No. 4	17 < 21 < 23 <	A B C	- 0 0	- 27 < 47 <
SMA 490W	365 or more	355 or more	335 or more	325 or more	490 - 610	≤16 16 < 40 <	No. 1A No. 1A No. 4	15 < 19 < 21 <	A B C	- 0 0	- 27 < 47 <
SMA 570W	460 or more	450 or more	430 or more	420 or more	570 - 720	≤16 16 < 20 <	No. 5 No. 5 No. 4	19 < 26 < 20 <	-	-5	47 <

*: According to JIS Z 2241 (Method of tensile test for metallic materials)

**: When testing at lower temperature than those, it may be replaced by the test temperature.

Table 3-9: Chemical Components of Weather-Resistant Hot-Rolled Steel for Welded Structure (steels whose yield point or proof stress does not change regardless of plate thickness)

Steel Grade	Tensile test					Impact test	
	Yield point or proof stress (N/mm^2)	Tensile strength (N/mm^2)	Elongation			Test temperature** (°C)	Charpy absorption energy (J)
	Steel thickness (≤100mm)		Steel thickness (100mm)	Test piece*	Elongation (%)		
SMA 400 CW-H	235 or more	400 - 540	≦16 16 < 40 <	No. 1A No. 1A No. 4	17 < 21 < 23 <	0	47 <
SMA 490 CW-H	355 or more	490 - 610	≦16 16 < 40 <	No. 1A No. 1A No. 4	15 < 19 < 21 <	0	47 <
SMA 570 CW-H	450 or more	570 - 720	≦16 16 < 20 <	No. 5 No. 5 No. 4	19 < 26 < 20 <	-5	47 <

*: According to JIS Z 2241 (Method of tensile test for metallic materials)

**: When testing at lower temperature than those, it may be replaced by the test temperature.

Note: "-H" following the name of a steel grade is the symbol to distinguish it from the JIS standard material.

vii) Methodologies and Details of detailed design are described in IRC:24-2010 which shall be referred to. Most of them are as same as non-weathering steel.

viii) Specific treatments required for weathering steel are quoted from Specifications for Highway Bridges of Japan (Part II Steel Bridges).

Quoted from Specifications for Highway Bridges of Japan (Part II Steel Bridges)

- For the high strength bolts used on weathering steel, none is specified yet in JIS. However, the bolt used shall conform to the specifications for the high strength bolt of the same type specified in this clause (Clause 7.3.2 Bolts, Nuts and Washers).
- Regarding the filler material, a slip strength test was conducted on friction grip connections in which a filler of rolled steel for general structure was used, and it was confirmed that the connections had the

required slip strength irrespective of the grade of steel of the filler. Therefore, rolled steels for general structure may be used as described in this clause. However, when the base metal is a rolled weathering steel, as a general rule the same steel material should be used for the sake of rust and corrosion prevention. Likewise, note that there are cases where the material is restricted by factors other than the stress-transfer function of the joint. (Clause 7.3.15 Filler)

- Welding of weathering steel and common steel: Welding consumables possessing mechanical properties and toughness equal to or greater than the common base metal. (Clause 18.4.2 Welding Consumables)
- Welding of weathering steels: Welding consumables possessing mechanical properties, toughness and weatherproof performance equal to or greater than the base metal. (Clause 18.4.2 Welding Consumables)
- When falling under the following cases, low-hydrogen electrodes shall be used:
  i) When a weathering steel is welded (Clause 18.4.2 Welding Consumables)
- The division of usage for welding consumables is specified here. When a covered electrode is used for weathering steel or high-tensile steel, it is specified that only a low-hydrogen electrode shall be used in consideration of crack resistance. (Clause 18.4.2 Welding Consumables)
- Preheating: Previous research has revealed that it is better to use the weld cracking sensitivity composition (P_{CM}) as an index on preheating rather than the carbon equivalent (C_{eq}) that has conventionally been used. And in addition to the expansion of applicable steel plate thickness in the revision on 1996, the applicable thickness of weathering steels was also expanded to 100 mm in the previous revision. Correspondingly, it has become necessary to more accurately select the preheating condition in welding in order to prevent the delayed cracking due to hydrogen, so the provisions of preheating are reviewed on the basis of P_{CM} (Clause 18.4.4 Welding Procedure)
- Weathering steel is obtained after the quality of the steel itself is improved by adding elements of alloy to the steel, whereupon dense rust generated on the steel surface blocks off water and oxygen, thus restraining the corrosion that takes place afterward. However, the proper repetition of wetting and drying is required for the creation of dense rust, and the usage environment to exert the required performance is limited. Accordingly, in respective stages of work such as delivery and storage as well, care should be taken so that corrosive elements such as salt may not become attached to an excessive degree. Moreover, when foreign substances, such as mill scale, member marks, and stains that obstruct the formation of dense rusts, exist on the steel surface, the irregularity in the formation of rust arises to spoil the appearance. (Clause 18.10 Rust Prevention and Corrosion Prevention)

[Source: Specification for Highway Bridges of Japan (Part II Steel Bridges)]

5) Steel Orthotropic Deck

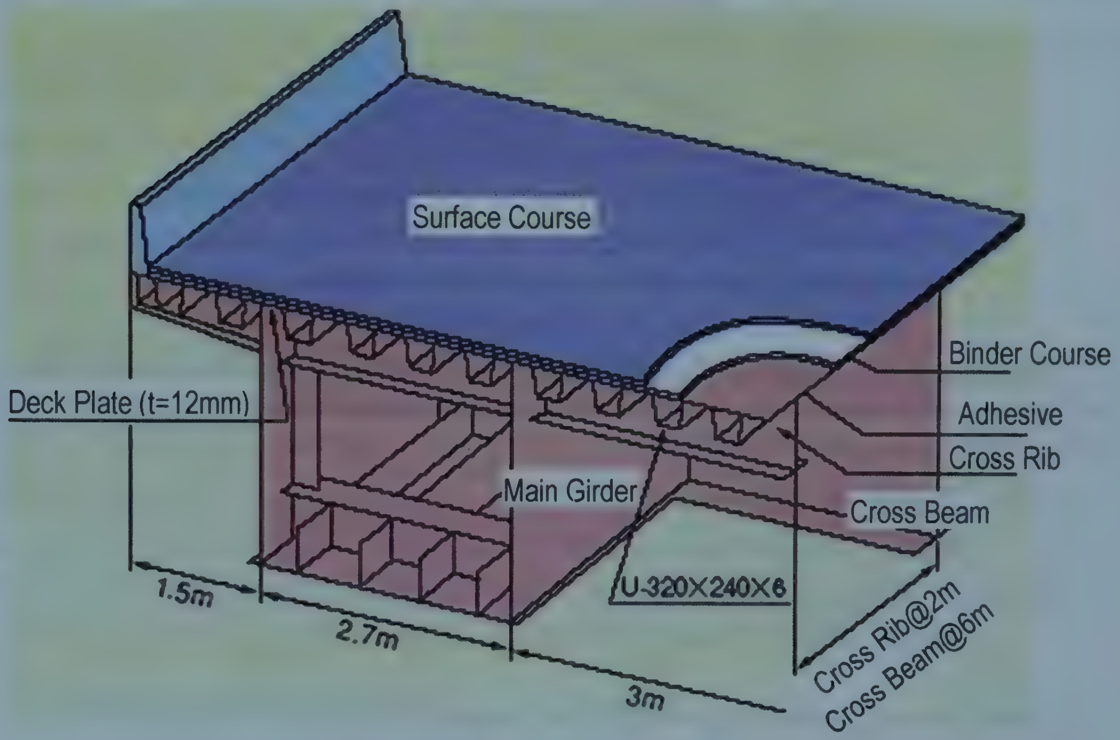
i) Steel orthotropic deck is utilized to increase span length without much increase of girder height. It can increase the span length twice as much in box girder with only about 30% increase of girder height if the main girder is steel box girder. It also reduces the superstructure dead load considerably (up to 50%) to reduce the force to substructure. Therefore, it is often applied to the bridges with strict restraints of girder height as well as substructure location, which can be seen in urban areas such as Railway Over Bridges, flyovers at grade-separated intersection, and river/canal crossing bridges in urban area.

ii) Steel orthotropic deck are applied for very long span bridges which have more than 200m, such as cable stayed bridges and suspension bridges for Mountain bridges. For other types of superstructure, it requires needs of reducing dead load and girder height, such as construction restraint of foundation with strong seismic force, and etc.

iii) Since the entire deck/slab is made steel plate strengthened by steel ribs, repetitive live load by heavy vehicles damages steel plate, ribs, and welded sections and leaves cracks caused by fatigue. Countermeasure for cracks caused by fatigue shall be included in detailed design of steel orthotropic deck.

iv) Main advantage of steel orthotropic deck is its light weight. It enable to extend the span length of the superstructure considerably compared to the superstructure with RCC/PSC deck slab. Therefore, it is applied to bridges with long spans with many constraints in constructing substructure or extremely long span.

v) The typical cross section of the steel box-girder bridge with steel orthotropic deck is shown in the figure below.



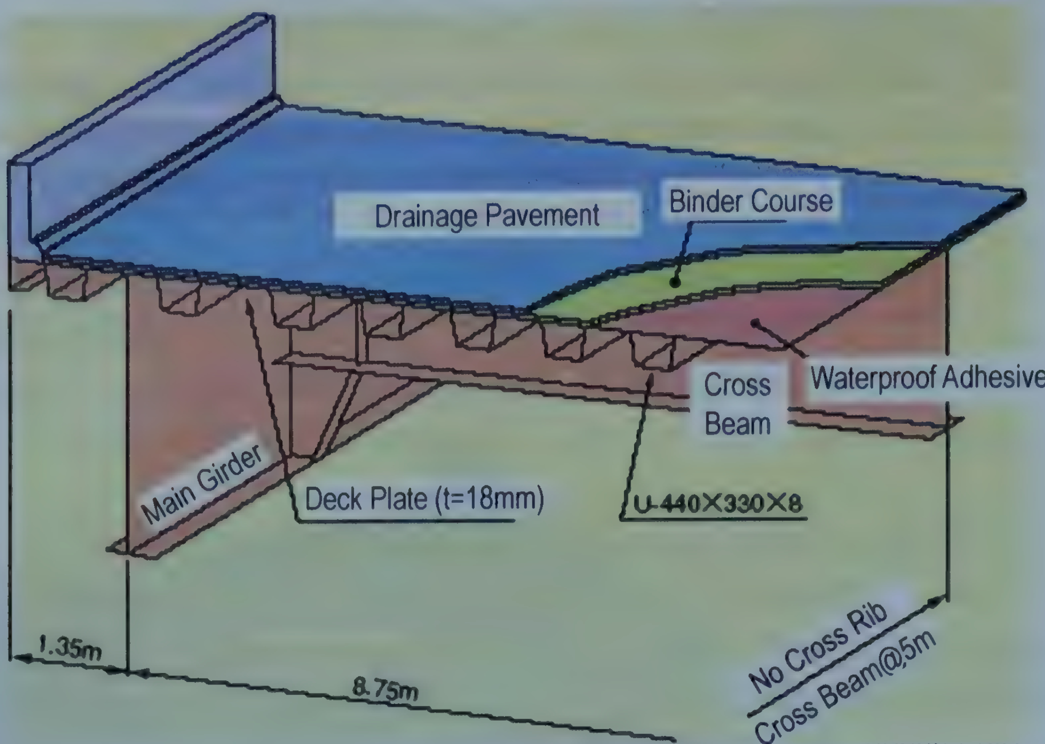
(Source: https://www.jasbc.or.jp/technique/document/)

Figure 3-1: Typical Cross Section of the Steel Box-girder Bridge with Steel Orthotropic Deck

vi) The disadvantages of steel orthotropic deck are complicated structure of many steel members (girder, rib, beam, etc.), and the large deformation of the slab caused by heavy vehicles. The former increases the procedure of fabrication and construction and makes the quality control difficult. It also makes the inspection and repair works difficult which leads the maintenance of the bridge considerably complicated and expensive. The latter requires frequent execution of inspection and repair works. These two main disadvantages contradicts each other and extremely difficult to balance each other with saved cost.

vii) There are some serious defects are found at the steel orthotropic decks due to fatigue caused by repetitive deformation, especially at the welding line between main deck and ribs. There shall be a consideration to improve workability of welding and also the quality control in fabrication.

viii) Recently, there is a simplified cross section of steel girder bridge with orthotropic deck. It replaced the box-girder with plate-girder and simplified the details of locations of ribs and beams. It also applied thicker steel plate for all the members, such as girder, beam, rib, and deck plate. It decreased the procedure of implementation and made it easier for inspection and repair.



(Source: https://www.jasbc.or.jp/technique/document/)

Figure 3-2: Typical Cross Section of the Steel Plate-girder Bridge with Simplified Steel Orthotropic Deck

ix) There in not any IRC Code, specification, manual, and guideline for designing steel orthotropic deck. However, IRC:24-2014 shall be referred to as steel structure. And also there are descriptions in Specifications for Highway Bridges of Japan (Part II Steel Bridges), Subchapter 9.4 'Orthotropic Steel Deck', as follows,

Quoted from Specifications for Highway Bridges of Japan (Part II Steel Bridges)

9.4 Orthotropic Steel Deck

9.4.1 Scope of Application

This clause shall be applied to the design of an orthotropic steel deck made of deck plate, stiffened through the use of longitudinal and transverse ribs, and paved.

The scope of application of this clause is specified. Conventional buckle plate and grating can also be regarded as a kind of orthotropic steel deck, but they aren't covered in this clause.

9.4.2 Design in General

When the orthotropic steel deck acts as part of a main girder, the design shall be made in accordance with the following stipulations.

1) The orthotropic steel deck shall be checked for safety from the following two actions:
   i) Action as part of a main girder
   ii) Action as a deck and floor system
2) When designing an orthotropic steel deck, it shall be checked for safety in the case where the two actions given in 1) are considered at the same time. In this case the stress shall be calculated for each action in the most adverse load state of the orthotropic steel deck, and a check shall be made for the sum of the stresses thereby calculated in accordance with the allowable stress provided in Table 9.4.1.
   For a weld zone or a high-strength bolted joint, a check shall be made against the allowable stress given in Clause 3.2.3, the allowable stress due to the allowable force increased by 40 percent, and the allowable force.

Table9.4.1 Allowable Stress in case Main Girder Action and Deck and Floor System Action are Considered at the Same Time

(N/mm^2)

Steel plate thickness (mm) \ Type of steel	SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
40 or less	195	260	295	355
Over 40 and up to 7 5	175	245	275	345
Over 7 5 and up to 100			265	335

(2) The design of an orthotropic steel deck as a deck and floor system shall be made in accordance with the stipulations of the following 1) to 4):

1) The live load shall be in accordance with Clause 2.2.2 of "Part I: Common."

2) The impact factor i shall be as follows.

   i) Longitudinal rib: i = 0.4

   ii) Transverse rib: i = 20/(50 + L)

   where L: Span (m) of transverse rib

3) For a bridge designed using the B-live load, the section force used in the design of a transverse rib shall be the section force calculated in 1) and 2), multiplied by the overdesign factor calculated through Equation 9.4.1.

$$\left.\begin{array}{ll} k=k_0 & (L \leq 4) \\ k=k_0-(k_0-1)\times(L-4)/6 & (4 < L \leq 10) \\ k=1.0 & (L > 10) \end{array}\right\} \cdots\cdots\cdots\cdots\cdots\cdots (9.4.1)$$

$$\begin{array}{ll} \text{with the condition of } k_0=1.0 & (B \leq 2) \\ k_0=1.0+0.2\times(B-2) & (2 < B \leq 3) \\ k_0=1.2 & (B > 3) \end{array}$$

where

L: Span (m) of transverse rib,

B: Transverse rib spacing (span of longitudinal rib) (m)

4) For a bridge designed using the A-live load, the section force used in the design may be taken to be the section force calculated in 1) and 2), reduced by 20 percent.

(3) When designing an orthotropic steel deck, due consideration shall be given to the effects of fatigue due to the passage of large vehicles.

(1) An orthotropic steel deck is made of deck plate stiffened by longitudinal and transverse ribs and supported by stringers, crossbeams or other floor structures, or main girders. In this case it is advantageous to make the transverse ribs double as the crossbeams of the floor system, or to use the deck plate and longitudinal ribs as the flange of a main girder and make them carry a portion of the main girder stress. Therefore, this clause is provided on the assumption of designing an orthotropic steel deck considering its action as a deck and floor structure and as part of a main girder independently of each other.

1) For the action as part of a main girder, the orthotropic steel deck may be designed as the flange of the main girder stiffened by longitudinal ribs. In this case the live load is the L-load given in Clause 2.2.2 of "Part I: Common," and the impact factor is taken to be the same value for the main girder. Depending on the circumstances, some structural types do not have this action. In such cases the design may be made according to Clause (2) only as a deck and floor system.

   For the action as a deck and floor system, the deck may be designed regarding it as a plate-and grid structure or orthotropic plate as described in the commentaries to Clause (2). In this case the deck plate offers the action as a plate directly supporting the wheel load. However, when a small plastic deformation occurs in the deck plate, it is generally thought that the deck plate will act as a membrane instead of acting as a plate. Therefore, the load carrying capacity of deck plate is usually much larger than the value calculated based on the plate theory. For a deck plate of the minimum thickness provided in Clause 9.4.5 or more, it isn't necessary to make a check for stresses as a plate directly supporting loads.

2) The most adverse load states with respect to the respective actions of i) and ii) aren't always equal, and the L-load and T-load are different in natures. Therefore, when checking the sum of the stresses it will be necessary to deliberate the definition of load to begin with, if the most adverse load state has to be considered in the exact sense. Additionally, the most adverse load state has to be searched for in each part of the orthotropic steel deck. In order to eliminate such trouble, it is provided that the check may made for the simple sum of the respective most adverse cases of the actions of 1) and 2).

In the design of a joint between orthotropic steel decks as well, the check may be made against the allowable force and allowable stress of a weld zone and fastening steel provided in Clause 3.2.3, increased by approximately the same amount as the value shown in Table 9.4.1.

(2) For the action as a deck and floor system, the orthotropic steel deck shall be designed regarding it as a plate-and-grid structure or an orthotropic plate. When applying these theories, the rib spacing, number of ribs used, torsional rigidity and the like shall be considered.

When designing the transverse ribs of the orthotropic steel deck of a bridge designed using the B-live load, it is provided to establish an overdesign factor of the section force in the transverse rib if the transverse rib spacing is great. Due to this overdesign factor, the sectional force in the transverse rib will increase if the transverse rib spacing is great. As measures in the design for this increase, the plausible method is to increase the material strength of the transverse rib, enlarge the cross-section or place the transverse ribs as densely as possible. In this case, a local deformation in the deck plate can cause pavement cracking and affect the durability of the orthotropic steel deck. In view of these effects, it is desirable to employ the method of placing the transverse ribs as densely as possible so as to increase the rigidity of the deck.

For a bridge designed using the A-live load, there is no need to consider the above-mentioned overdesign factor to the same effect as Clause 9.2.4, so it is provided that the sectional force due to the T-load (including impact) may be reduced by 20 percent.

On an orthotropic steel deck, the spans of the transverse and longitudinal ribs making up it are generally close, and the impact factor for T-load takes a value close to 0.4 when calculated according to Clause 2.2.3 of "Part I: Common." Therefore, it is provided to use 0.4 for the impact factor of the longitudinal rib. Lately, for the transverse rib a fairly large span has come to be used. Therefore, it is provided to use the impact factor calculated according to Clause 2.2.3 of "Part I: Common." The suggestions for the design of longitudinal and transverse ribs are enumerated as follows:

a) As a point to check when designing a longitudinal rib, it is good to take the points A and B shown in Figure C.9 .4.1. Point B is the center of the span of the longitudinal ribs situated near the center of the span of the transverse ribs. At this point, assuming an elastic deformation of the transverse ribs, calculate the longitudinal ribs as a plate-and-grid structure or an orthotropic plate structure and check for the positive bending moment. Point A is related to the longitudinal rib at a support of a transverse rib. At this point the elastic deformation of the transverse rib is small. So, regard the transverse rib as a rigid support, calculate the longitudinal rib as a continuous beam and check for negative bending moment.

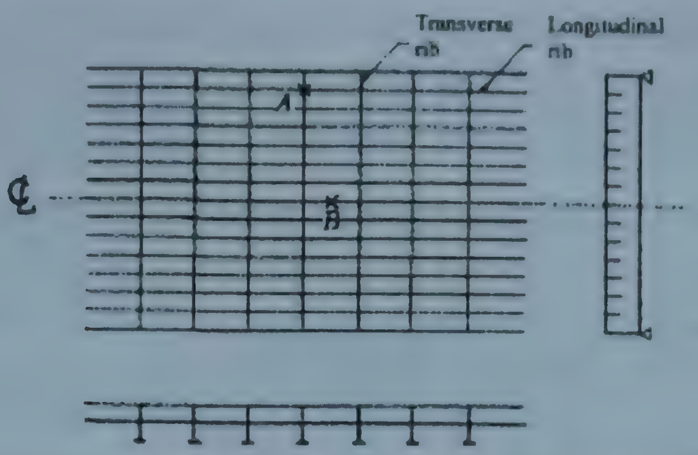


Figure C.9.4.1 Check Points when Designing Longitudinal Ribs

b) When designing a transverse rib, it may be calculated using the theory of plate-and-grid structures or orthotropic plate structures, assuming it is simply supported by supporting girders. The checkpoint in this case is the center of the span of the transverse rib and a check is made for positive bending moment. When the transverse rib continues and constraints at the support are conceivable, it is allowable with consideration of the constraint conditions to calculate the positive bending moment as a simple support and take, for example, ± 80 percent of it as the bending moments in the span and at the support.

c) At the girder end, the orthotropic steel deck loses its longitudinal continuity. Therefore, it is desirable to design the transverse rib there so that it can endure the T-load on its own.

d) When a transverse rib of particularly great bending stiffness is inserted between ordinary transverse ribs, it shall be especially noted that stresses concentrate at this transverse rib. In such cases it is good to design it to be able to endure the T-load by itself.

(3) Since an orthotropic steel deck receives wheel loads directly, due consideration shall be given in its design to the effects on its fatigue of the repeated passage of large automobiles. To ensure the fatigue durability of an orthotropic steel deck, its structural detail shall be determined in accordance with 9.4.7, and the "Fatigue Design Guidelines for Steel Highway Bridges" (Japan Road Association) 5l are also available as reference. The construction shall be according to the related Chapter 18, respectively.

9.4.3 Load Distribution by Pavement

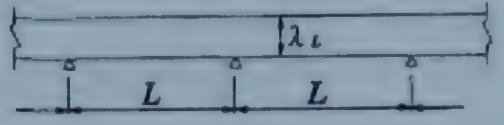
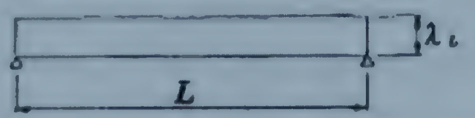
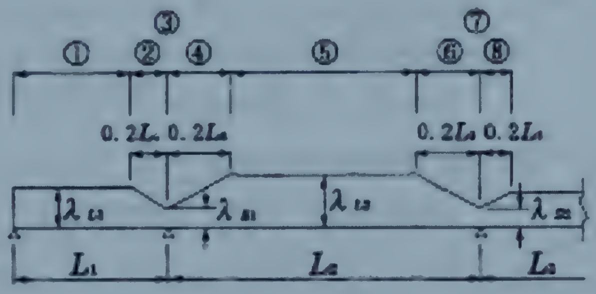
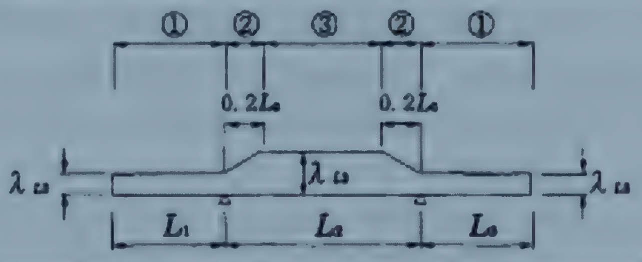
For the wheel load imposed on the deck plate of an orthotropic steel deck, the load distribution by the pavement shall not be considered.

On an orthotropic steel deck, an asphalt pavement 60 mm to 80 mm thick is generally constructed over the deck plate. With such a pavement it is known that the asphalt hardens in the winter and the pavement becomes fairly effective in terms of load distribution. However, in the summer the pavement does not help much in load distribution because the asphalt softens. Therefore, it is provided not to consider the action of live load distribution by the asphalt pavement on the orthotropic steel deck.

9.4.4 Effective Width of Deck Plate with respect to Deck or Floor System Action

The half effective width of the deck plate as a flange of a longitudinal rib or transverse rib shall be calculated through Equation 9.4.2, and its application shall be in accordance with Table 9.4.2.

Table 9.4.2 Effective Width of Deck Plate with respect to Deck or Floor System Action

Member		Section (portion)	Half effective width		Remarks
			Symbol	Equivalent span length	
Longitudinal rib			λ_L	$0.6L$	
Transverse rib	Simple support	①	λ_L	L	
	Continuous support	①	λ_{L1}	$0.8L_1$	 ③ ⑦ ① ② ④ ⑤ ⑥ ⑧ 0.2L_1 0.2L_2 0.2L_2 0.2L_3 λ_{L1} λ_{R1} λ_{L2} λ_{R2} L_1 L_2 L_3
		⑤	λ_{L2}	$0.6L_2$	
		③	λ_{R1}	$0.2(L_1+L_2)$	
		⑦	λ_{R2}	$0.2(L_2+L_3)$	
		② ④ ⑥ ⑧	Linear change using effective width at both ends		
	Cantilever portion	①	λ_{L1}	$2L_1$	 ① ② ③ ② ① 0.2L_2 0.2L_2 λ_{L1} λ_{L2} λ_{L1} L_1 L_2 L_1
		③	λ_{L2}	L_2	
		②	Linear change using effective width at both ends		

$$
\left.\begin{array}{ll}
\lambda = b & \left(\frac{b}{l} \leqq 0.02\right) \\
\lambda = \left\{1.06 - 3.2\left(\frac{b}{l}\right) + 4.5\left(\frac{b}{l}\right)^2\right\} b & \left(0.02 < \frac{b}{l} < 0.30\right) \\
\lambda = 0.15l & \left(0.30 \leqq \frac{b}{l}\right)
\end{array}\right\} \cdots\cdots\cdots (9.4.2)
$$

where

J: Half effective width (mm) of deck plate

2b: Spacing (mm) of longitudinal or transverse ribs

For a closed section longitudinal rib, these dimensions shall be as shown in Figure9.4.1.

/: Equivalent span length (mm)

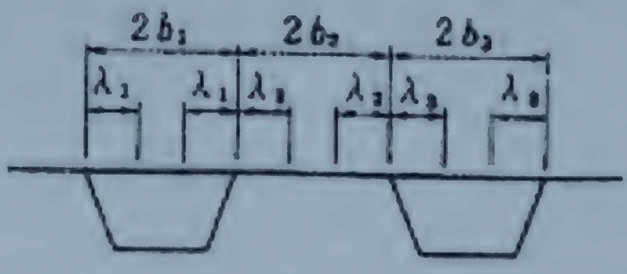


Figure 9.4.1 Spacing of Closed Section Longitudinal Ribs

Part of the deck plate effectively acts as the flange of a longitudinal or transverse rib. The effective width J varies with the load conditions, the ratio of rib spacing to span length, the support conditions, the cross-sectional configuration of the rib, the boundary conditions of the deck plate as the flange of a rib, and others, but the effects of the first three factors predominate.

Under the conditions of this section, the concept of equivalent span length is introduced to represent the support conditions in the same way as Clause 11.3.5. The bending moment becomes maximal at the center of the span and support. Considering concentrated loads as an object, the effective width at those positions is given by an equation that represents it as a function of the ratio of rib spacing to span length. The variations of effective width in the direction of the span are approximated by a line.

9.4.5 Minimum Thickness of Orthotropic Steel Deck Plate

(1) The thickness, t (mm), of the orthotropic steel deck plate shall be the value calculated through Equation 9.4.3 or more.

$$\left.\begin{array}{ll} \text{Roadway portion:} & t = 0.037 \times b, \text{ (B-live load)} \\ & t = 0.035 \times b, \text{ (A-live load)} \\ & \qquad \text{with the condition of } t \geq 12\text{mm} \\ \text{Sidewalk portion acting as part of} & \\ \text{main girder:} & t = 0.025 \times b, \\ & \qquad \text{with the condition of } t \geq 10\text{mm} \end{array}\right\} \quad \text{..............................(9.4.3)}$$

where

b: Longitudinal rib spacing (mm)

(2) When a closed section longitudinal rib is used immediately under locations that receive the wheel loads of large automobiles constantly, the thickness of the deck plate shall be 16 mm or larger in principle, without referring to Clause (1).

(1) The thickness of the deck plate shall be determined considering at least strength, rigidity, constructability and the like. According to the experience to date, if the rigidity of the deck plate is insufficient the pavement will be adversely affected. With the currently available paving techniques, it is considered appropriate to restrict the deflection of the deck plate due to wheel load to 1/300 of the longitudinal spacing or less. The equations in this clause were determined according to this thought, meaning t = 0.037 x bis for the B-live load as a wheel load including impact, and t = 0.035 x b is for the A-live load as a wheel load including impact, respectively.

When the sidewalk portion of the deck plate of an orthotropic steel deck carries a part of the main girder action, the thickness of the deck plate will have no large effect on the pavement, unlike the case of the roadway portion of the deck plate. However, there are many cases where the deck plate is partially subjected to a considerable compressive stress as the flange of a girder. With the same plate thickness, the welding distortion is greater for larger longitudinal rib spacing. Considering these and referring DIN and other standards, it has been decided to provide that the thickness of the deck plate in its sidewalk portion shall bet= 0.025 x b.

The longitudinal ribs are all attached to the deck plate by welding. With a thin deck plate, the welding distortion is great, which is undesirable in terms of strength and pavement. The amount of welding distortion varies with the welding method, the extent of welds, the type and spacing of the longitudinal ribs, the deck plate thickness and other factors. Of these, the thickness of the deck plate is considerably dominant. Thus it is provided for deck plate thickness that the minimum is 12 mm and 10 mm in the roadway and sidewalk portions, respectively.

For the sidewalk portion of a deck plate that does not act as part of a main girder, the deflection due to live load is small and the welding distortion of the deck plate has a small effect on strength and pavement. Therefore, the thickness may be taken to be 8 mm according to the provision of minimum thickness of steel in Clause 4.1.4.

(2) In recent years, some damage cases of the orthotropic steel decks of existing bridges have been reported. These damage cases were caused by fatigue cracks running through a deck plate from the portion where a closed section longitudinal rib (U rib) was welded onto the deck plate immediately under a location that receives the constant wheel loads of large automobiles. According to the reports of the previous investigations and studies, most of the cracks developed in deck plates with the minimum plate thickness of 12 mm, and fatigue durability can be enhanced by increasing the thickness of deck plates8). Against this backdrop, although factors in creating cracks and their growth and behavior are not completely understood, this stipulation specifies a method for designing deck plates as a measure to enhance the fatigue durability of orthotropic steel decks using close-section longitudinal ribs. It states that a deck plate shall be designed referring to the structural details described in the "Fatigue Design Guidelines for Steel Highway Bridges" (Japan Road Association)5) and that the thickness of deck plates immediately under locations that receive constant wheel loads of large automobiles shall be 16 mm or larger in principle. Since this stipulation is made on the premise of the spacing of the longitudinal ribs as calculated on the assumption of a deck plate thickness of 12 mm from Equation 9.4.3, this assumption shall be satisfied. When an open section rib such as a valve plate and a flat plate rib is used, the thickness of deck plates for roadways shall be determined from Equation 9.4.3.

9.4.6 Minimum Thickness of Longitudinal Rib

The minimum thickness of the longitudinal rib shall be 8 mm. However, when the environment is noncorrosive or corrosion is properly considered, the minimum thickness of a closed section longitudinal rib may be taken to be 6 mm.

The minimum thickness of steel used for a longitudinal rib is 8 mm according to the provisions of Clause 4.1.4. With an orthotropic steel deck using closed section longitudinal ribs 8 mm in thickness, there is generally a considerable margin for the stress in the longitudinal rib, so there are cases where the performance of the closed section longitudinal rib cannot fully exploited. Thus the minimum thickness of a closed section longitudinal rib is permitted up to 6 mm only in a place where the environment is non-corrosive and there is little fear of thickness reduction due to corrosion, or only if due attention is given to rust and corrosion prevention as well as the routine inspection and maintenance system of the longitudinal ribs.

9.4.7 Structural Details

(1) The orthotropic steel deck shall have a structure in which the distortion due to welding is minimal.

(2) The connection between longitudinal and transverse ribs shall have a structure that can surely transfer shear forces from the longitudinal rib to the transverse rib. Except for special cases it is desirable that the longitudinal rib continues through the web of the transverse rib.

(3) When main girders or stringers are placed in the roadway portion, attention shall be paid to suppression of cracking in the pavement above the web.

(4) The joint of a longitudinal rib shall, generally, be a high-strength bolted joint.

(5) When connecting deck plates by high-strength bolts, consideration shall be given to the effects of the connecting plates, bolts and other projecting objects on the pavement.

(1) A number of welds concentrate at the intersection of longitudinal and transverse ribs and on the deck plate, and therefore the welding distortion is likely to be great. Therefore, due attention shall be paid to the design and construction.

(2) When the longitudinal rib loses its continuity at the web of a transverse rib, such a structure will have problems in strength, particularly in terms of fatigue strength. Therefore, this provision has been established.

Where the longitudinal rib passes through a transverse rib, the intersection between them must be welded to ensure the transfer of shear forces from the longitudinal rib to the transverse rib.

(3) For the roadway portion of the orthotropic steel deck, a minimum thickness of deck plate is provided as a limit of deflection due to wheel load in Clause 9.4.5 in order to prevent adverse effects on the pavement. Moreover, when main girders or stringers are placed in the roadway portion it is necessary to pay attention to the suppression of cracking in the direction of the bridge axis in the pavement directly above the web. To suppress this cracking in the pavement, it is desirable to give consideration to the design to ensure that the ordinary running position of wheel load does not come directly above the web. Or, it is good to design the spacing between a main girder or the like and the longitudinal rib and the span and rigidity of the longitudinal ribs so that the deformation of the pavement over the web does not become large. As a structural design guide for this case, it is desirable to take the radius of curvature of the deck plate over the web caused by live loads at 20 m or more (take the rigidity of the pavement at an Young's modulus of 2 x 103 N/mm^2 and consider the composite effects between the pavement and orthotropic steel deck), referring to Reference 6).

For a general orthotropic steel deck, the references 5) and 7) can be used as a reference when setting the dimensions and spacing of longitudinal ribs and the spacing of transverse ribs. Figure C.9.4.2 represents the relationship between the rigidity of a longitudinal rib and the transverse rib spacing in a simple form for the case where the radius of curvature of the deck plate caused by wheel load is taken at 20 m or more and the amount of deflection between longitudinal ribs is taken at 0.4 mm or less7>. This clause does not prohibit measures other than mentioned above, if consideration is given to the pavement and structure to suppress pavement cracking.

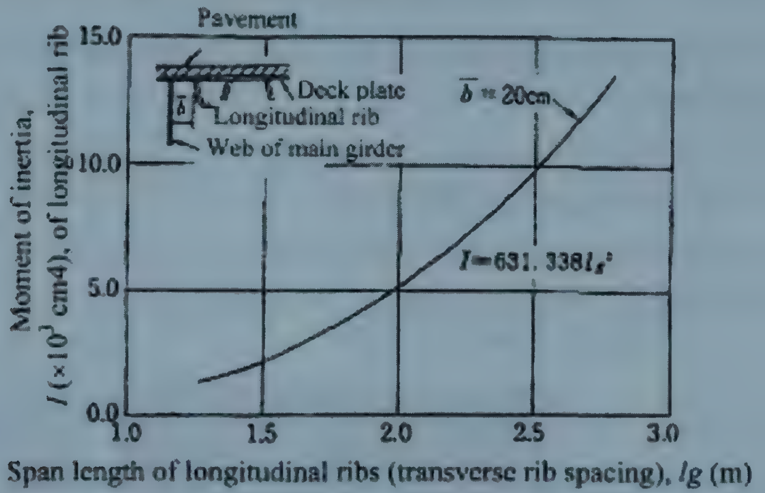


Figure C.9.4.2 Relationship between Rigidity of Longitudinal Rib and Span Length of Longitudinal Ribs7> (transverse rib spacing)

(4) For field joint of flat plate and valve plate ribs, high-strength bolted joints have so far been used. And for field joint of closed section, high-strength bolted and welded joints have so far been used. However, the fatigue strength of a butt-welded joint with backing steels turned out to be low (Figure C.9.3.1). Therefore, as a rule a high-strength bolted joint shall be used. For the details of the joint structure of an orthotropic steel deck, it is good to use "Fatigue Design Guidelines for Steel Highway Bridges (Japan Road Association) 5> as a reference.

(5) If there are splice plates, bolts or other projecting object on the top surface of the deck plate, pavement construction and durability may be adversely affected, so attention must be paid to these objects6), 7). When designing bridge deck pavement, "Technical Standards for Pavement Structure" (Notice of urban and regional development director and road Director) shall be obeyed. For the treatment of lifting pieces, jigs and other projecting objects after cutting them off, the related provisions of Chapter 18 shall be obeyed.

[Source: Specification for Highway Bridges of Japan (Part II Steel Bridges)]

(4) Hybrid Superstructure

1) General

i) Hybrid structures with steel and concrete (both RCC and PSC) is developed to take advantage of characteristics of both steel and concrete and rationalize the bridge structure in aspects of economy efficiency, durability, and safety. There are two types of hybrid structures which are composite structure and mixed structure.

ii) Composite structure is defined that the cross section of the structure combines different materials as steel and concrete. The typical structure of composite bridge is the steel plate girder bridge with RCC deck connected firmly by friction or mechanical shear connectors. The details are explained below.

iii) Mixed structures defined by JSCE (Japan Society of Civil Engineers) is follows,

> A mixed structure is a structure where more than two different types of members such as steel members, concrete members and various composite members are connected. In mixed structures, constituting members can be chosen depending on usage conditions and objectives, and consequently the strength, stiffness and ductility of different types of members are effectively utilized. Therefore, mixed structures have attracted attentions and actively been applied. Mixed structures also give design engineers large freedom of structural planning and design. The performance of mixed structures depends on that of connections which should be designed as an independent member in a structure.
>
> Source: Basic Theory and Design of Hybrid Structures -2019-, JSCE

2) Composite Superstructure

i) Types of composite superstructure:

There are approximately five types of composite structures defined by JSCE such as,

- Composite girder bridge (plate girder of box girder)
- Composite truss bridge
- Corrugated steel web bridge
- Hybrid truss bridge
- Composite slab bridge

Corrugated steel web bridge

Composite truss bridge

ii) IRC:22-2015 defines following composite superstructures,

- Composite girder bridge (Plate girder of box girder)
- Filler Beam Decks
- Precast Slab on Steel Beams

IRC;22-2015 shall be referred to for detailed design for above 3 superstructures.

iii) Suitable span lengthen by composite superstructure is shown on table below

Table 3-10: Suitable Span Length by Composite Superstructure

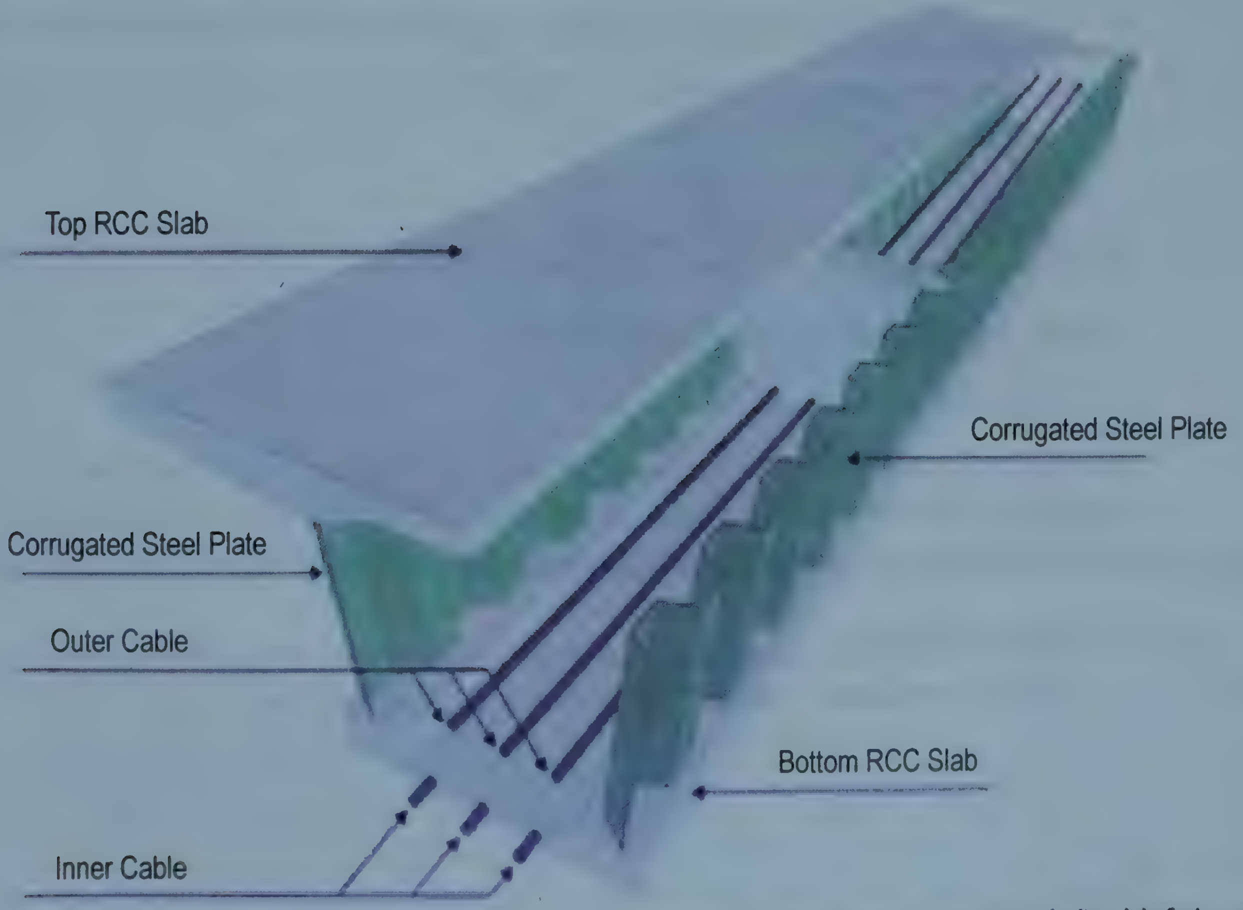
Type of superstructure	Span length
Composite girder bridge (plate girder)	30 to 80 m
Composite girder bridge (box girder)	40 to 100 m
Composite truss bridge	80 to 150 m
Corrugated steel web bridge (box cross section)	40 to 150 m
Corrugated steel web bridge (extra-dosed bridge)	50 to 190 m
Hybrid truss bridge (box cross section)	30 to 120 m
Hybrid truss bridge (extra-dosed bridge)	120 to 200 m
Filler Beam Decks	10 to 40 m

iv) Corrugate Steel Web Bridge is a type of superstructure which has corrugate steel plate for web of PSC box-girder. It has both inner and outer prestressed cables in the cross section. Corrugate steel web reduces the superstructure dead load and enables to extend the span length. It can be applied not only to PSC box-girder, but also to extra-dozed or cable stayed bridges.

v) Corrugate steel plate increases the prefabrication ratio of the material and reduces the construction procedure at site. It shortens the construction period as well as increases the workability on the site.

vi) Corrugate steel plate reduces the stiffness of bridge axial direction of superstructure and enables to transfer the prestress force more efficiently than ordinal PSC box-girder. It also resist to the shear force and does not require stiffener. The simple structure of this superstructure is shown on below figure.

vii) Details of design methodology and procedure is demonstrated below after the design methodology and procedure of composite members. It is quoted from Design Standard of NEXCO.



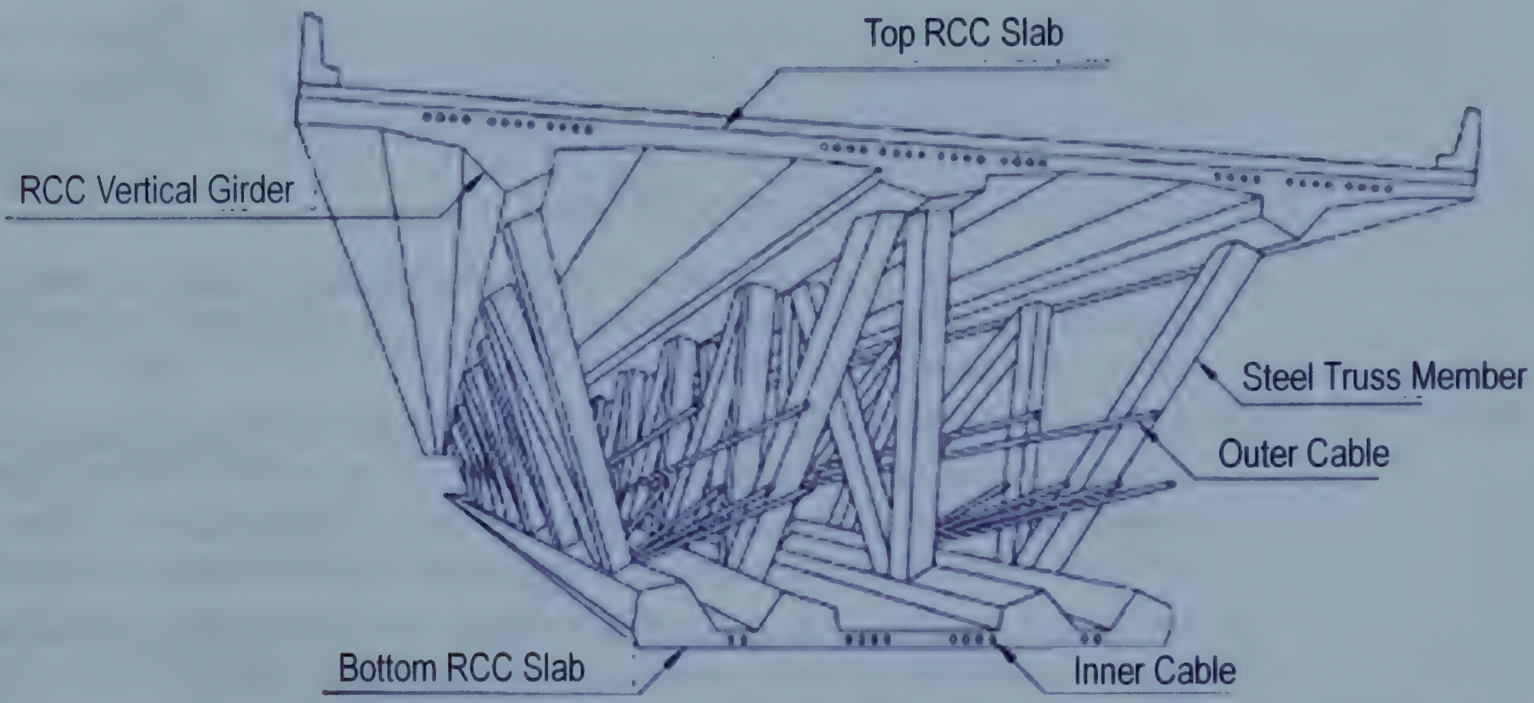
(Source: https://www.pcken.or.jp/techinfo/newtech/)

Figure 3-3: Structure of Corrugate Steel Web Superstructure

viii) Composite Truss Bridge is a type of superstructure which has steel truss for web of PSC box-girder. It has both inner and outer prestressed cables in the cross section. Steel truss reduces the superstructure dead load and enables to extend the span length. It can be applied not only to PSC box-girder, but also to extra-dozed or cable stayed bridges.

ix) Steel truss increases the prefabrication ratio of the material and reduces the construction procedure at site. It shortens the construction period as well as increases the workability on the site.

x) Dead load of composite truss superstructure does not change as much as other types of superstructure when the height of the girder increases. This advantage permits high girder without large increase of dead load and extends the span length.



(Source: https://www.pcken.or.jp/techinfo/newtech/)

Figure 3-4: Structure of Composite Truss Superstructure

xi) For reference Chapter 5, Basic Theory and Design of Hybrid Structures -2019-, JSCE is quoted as follows,

Quoted from Basic Theory and Design of Hybrid Structures -2019-, JSCE

Chapter 5 Design of Composite Members

5.1 Introduction

The design specification, properties of steel, concrete and FRP, and theory of composite structures are explained in the previous chapters. Chapters 5 and 6 describe basics and important points on design of composite members (composite girders, composite plates and composite columns) and mixed structures.

5.2 Composite girder

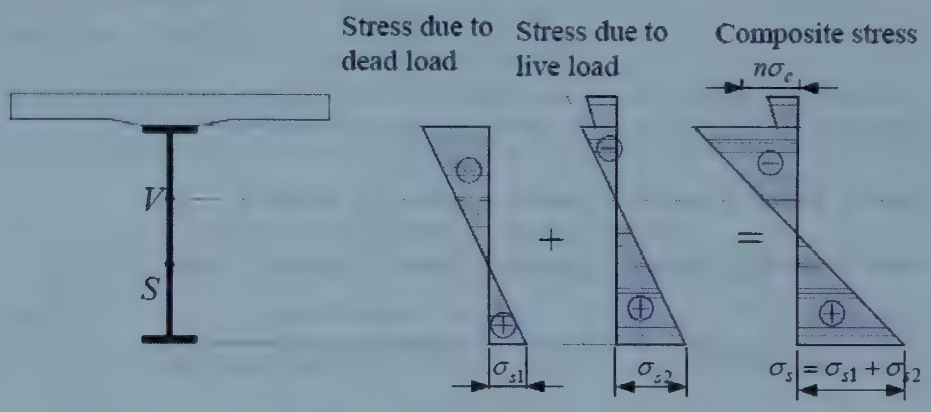
5.2.1 Simple composite girders and continuous composite girders

Composite girder is one of the bridge structures where the steel girder and the concrete slab are connected by using headed studs and so on. Composite girder uses concrete with high resistance against compression at the compressive side of the section and uses steel with high resistance against tension at the tensile side, rationally utilizing the material properties of both concrete and steel. It was developed in Europe and USA in the 1940s, stud welding was then developed and design was thought rational, it prevailed in the world in the 1950s. As concrete slab works as part of the girder in the composite girder, it is said that steel amount can be reduced by 20% of the plate-girder bridge1). The girder height can be also lowered.

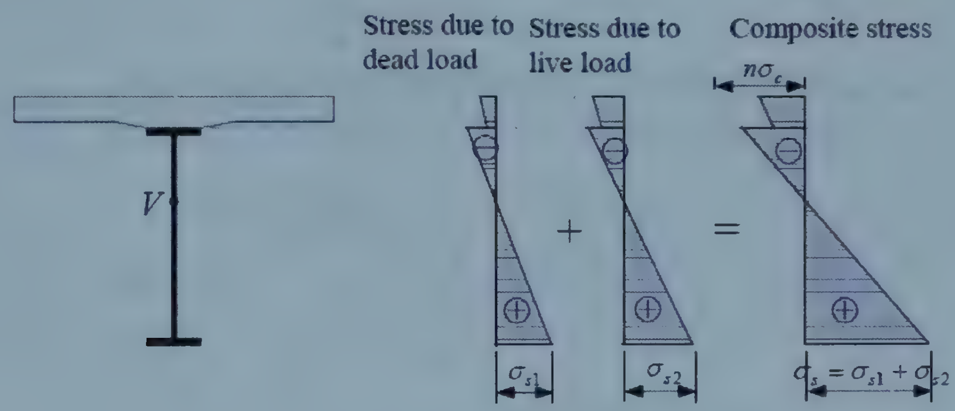
Composite girders are classified into the composite girder for live load and the composite girder for dead and live loads, depending on erection methods. Fig.5.2.1 shows the stress conditions of two types of composite girders due to dead loads, live loads and the total loads.

The composite girder for live load is that the concrete slab is installed on the steel girder supported on the bearings at both ends. Dead loads due to steel girders and concrete slab are resisted by the steel girder and post-dead loads and live loads are resisted by the composite girder. Most of the composite girder bridges are this type.

On the other hand, the composite girder for dead and live loads is that the concrete slab is installed while the steel girder is supported by temporary supports. After concrete is hardened, the temporary supports are removed. Therefore, both dead and live loads are resisted by the composite girder. However, when the ground is weak, temporary supports may sink and perfect composite action cannot be expected. Therefore, recently, this type of composite girder bridge is rarely applied. The simply supported composite girder of the above two types have positive bending moment on all the span. The concrete slab is, therefore, in compression and no consideration about tensile stress of concrete in longitudinal direction is necessary without the effects of creep, shrink and sink of supports.



(a) Live load composite girder

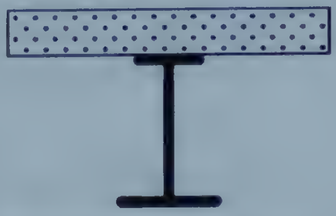
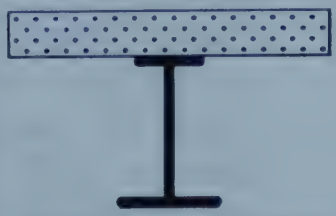
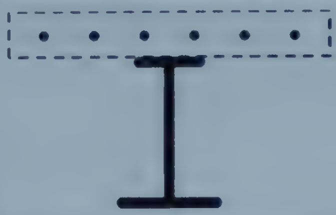


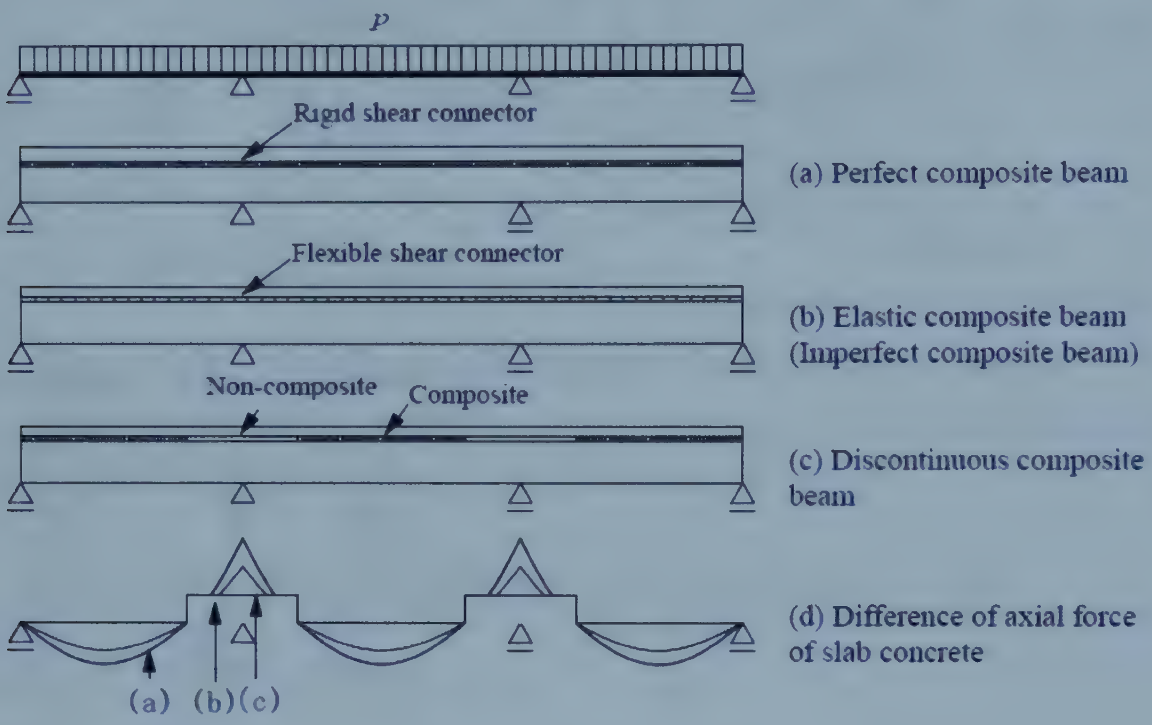
(b) Dead and live load composite girder

Fig.5.2.1 Stress distribution of live load composite girder and dead-live load composite girder

The continuous composite girder has negative bending moments at intermediate supports, which requires counter-measures to control cracks caused by tension in concrete slab. Specification for Road Bridges, II. Steel Bridge Part2), specifies the design methods to deal with negative bending moment at the intermediate supports, as shown in Table 5.2.1. When concrete slab is expected to resist tension, pre-stress forces must be introduced, which is complicated in design and construction. When concrete slab is not considered, stresses are calculated by assuming that only the steel girder and the reinforcement resist the negative bending moment. Specification for Road Bridges, II. Steel Bridge Part2), specifies that minimum amount of the reinforcement in the longitudinal direction should be 2% of the area of concrete and the ratio of total circumference length of the reinforcement to the cross- sectional area of concrete should be 0.0045mm/mm^2.

Table 5.2.1 Treatment of composite actions

Type of bending moment	Treatment of composite action		Illustration
Positive	Slab concrete is included in the girder section		
Negative	When concrete in tension is assumed effective,	Slab concrete is included in girder section.	
	When concrete in tension is neglected,	Steel bars can be included in the girder section.	


p
Rigid shear connector
(a) Perfect composite beam
Flexible shear connector
(b) Elastic composite beam
(Imperfect composite beam)
Non-composite
Composite
(c) Discontinuous composite beam
(d) Difference of axial force of slab concrete
(a) (b)(c)

(a) Perfect composite beam
(b) Elastic composite beam (Imperfect composite beam)
(c) Discontinuous beam

Fig.5.2.2 Comparison of axial force distribution of slab concrete for equally distributed load

There are two methods to reduce tensile axial force acting on the concrete slab on the negative bending moment area at intermediate supports, as shown in Fig.5.2.2: the elastic composite girder and the discontinuous composite girder. On the elastic composite, more flexible shear connectors compared with perfect composite girder is used in the whole span. On the discontinuous composite girder, the intermediate support part is non-composite and other part is composite. Both types aim to disperse the negative moments and to reduce stresses. There are some problems to be tackled in both types such as evaluation of composite degree with arrangement of shear connectors and length of non-composite part and so on.

5.2.2 Effective width of slabs

The concrete slab is composite with the steel girder on the conventional composite girder. When the composite girder is subjected to positive bending moment, the concrete slab works as part of compressive flange of the steel girder. Then, the compressive bending stress varies in the width-wise direction due to shear lag, as shown in Fig.5.2.3. This distribution depends on the width of the ratio b/ℓ of the slab width b to span length ℓ loading condition, support condition of the girder and so on. As it is difficult to reflect this on design, the effective width be is introduced to equalize it with maximum stress σ max. The effective width is used to calculate stress, deformation and ultimate strength of composite girders by elementary beam theory.

$$b_e = \frac{1}{\sigma_{\max}} \int_0^b \sigma(s)\,ds \qquad (5.2.1)$$

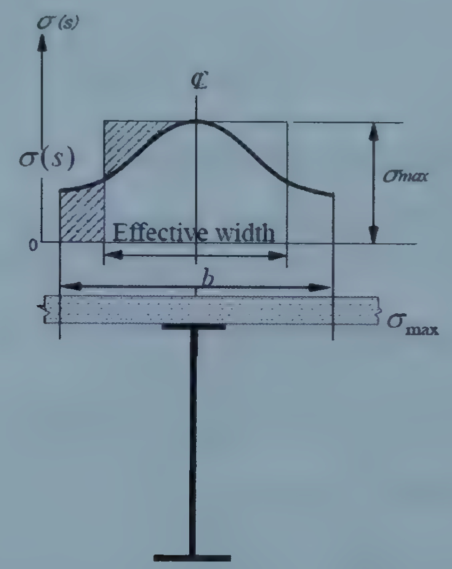


Fig. 5.2.3 Effective Width of Slab

The effective width specified in design guidelines or specifications must be used for an actual design. In Japan, the following method is used to set the effective width of the compressive flange of the composite girder (Fig.5.2.4).

$$b_e = \lambda_1 + \lambda_2 \qquad \text{without haunch}$$

$$b_e = \lambda_1 + \lambda_2 + b_s + b_0 \qquad \text{with haunch} \qquad (5.2.2)$$

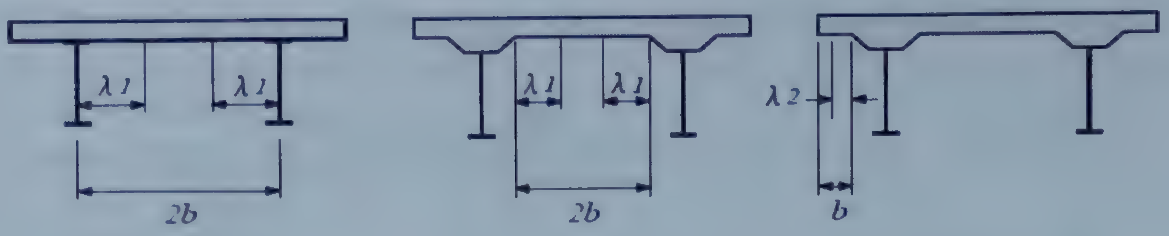
where,

$$\left.\begin{array}{ll} \lambda_1 \text{ or } \lambda_2 = b & \left(\frac{b}{\ell} \le 0.05\right) \\ \lambda_1 \text{ or } \lambda_2 = \left\{1.1\text{-}2\left(\frac{b}{\lambda}\right)\right\} b & \left(0.05 < \frac{b}{\ell} \le 0.30\right) \\ \lambda_1 \text{ or } \lambda_2 = 0.15\lambda & \left(0.30 \le \frac{b}{\ell}\right) \end{array}\right\} \qquad (5.2.3)$$

where,

b_s : horizontal width of haunch (If the angle is smaller than 45 degrees, it is set at 45 degrees.)
b_0 : width in the vicinity of haunch
λ_1 : effective width of one inner side of beam
λ_2 : effective width of one side of cantilever part
b : half of the distance between webs or length of cantilever part
l : span length of girder

When the slab is subjected to negative bending moments, the effective width can be obtained by the same method described above.



1) Center flange part 2) Cantilever part

(a) With haunch (b) Without haunch

Fig.5.2.4 Calculation for effective width

5.2.3 Stress and deformation of composite girders

In order to conduct the allowable stress method or to verify serviceability of the limit states design, elastic stresses and deformations of the composite section are calculated under the following conditions.

(1) It conforms to the Bernoulli-Euler assumption in the composite section.
(2) Tensile resistance of concrete is neglected.

When the steel section is large, the neutral axis is located generally in the steel section due to positive bending moment. The distance from the centroid of the composite girder to that of the steel girder and the concrete slab, moment of inertia of equivalent total section, stresses at the lower and upper flanges and upper edge of concrete slab, are obtained as follows (see Fig.5.2.5).

$$d_s = \frac{A_c}{nA_s + A_c} d_0 \qquad (5.2.4)$$

$$d_c = \frac{nA_s}{nA_s + A_c} d_0 \qquad (5.2.5)$$

$$I_v = I_s + \frac{I_c}{n} + A_s d_s^2 + \frac{1}{n} A_c d_c^2 \qquad (5.2.6)$$

Stress at upper edge of steel girder: $\sigma_{su} = \frac{M_1}{I_s} y_{su} + \frac{M_2}{I_v} y_{vu}$ (5.2.7)

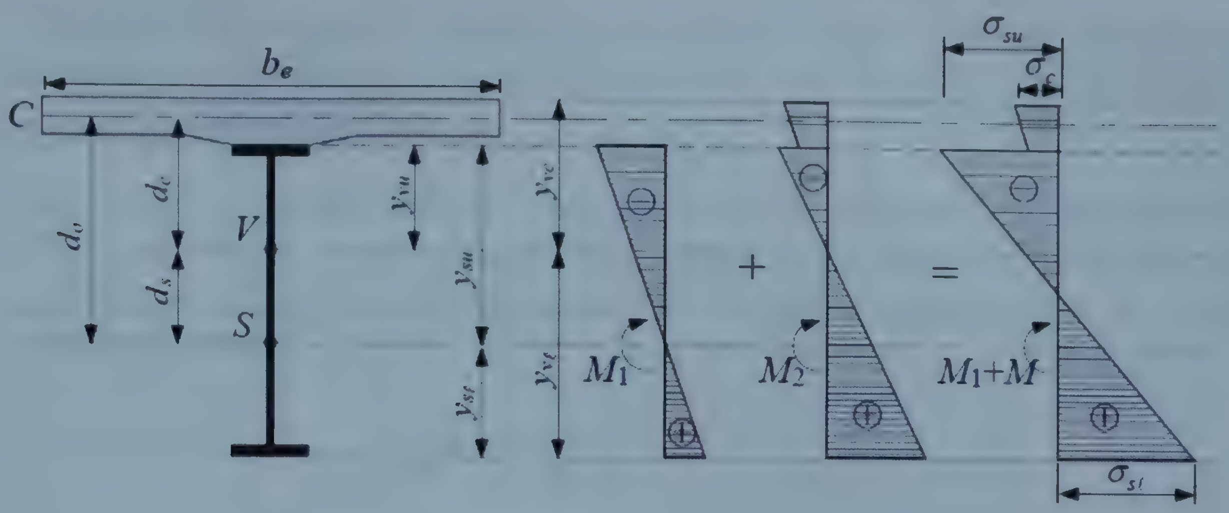
Stress at lower edge of steel girder: $\sigma_{s\ell} = \frac{M_1}{I_s} y_{s\ell} + \frac{M_2}{I_v} y_{v\ell}$ (5.2.8)

Stress at upper edge of RC slab: $\sigma_c = \frac{M_2}{nI_v} y_{vc}$ (5.2.9)

where,

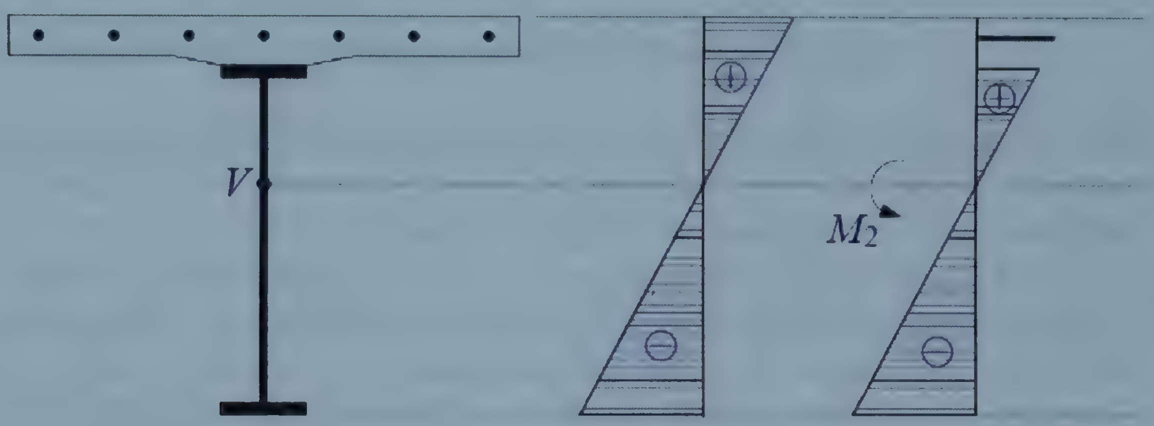
n : ratio of Young's modulus of steel to concrete (normally n=7)
A_c : cross-sectional area of concrete slab
A_s : cross-sectional area of steel girder
I_c : moment of inertia of concrete slab around centroid axis
I_s : moment of inertia of steel girder around centroid axis
y_{su} : distance from centroid axis of steel girder to upper edge of steelgirder
$y_{s\ell}$: distance from centroid axis of steel girder to lower edge of steelgirder
y_{vu} : distance from centroid axis of composite girder to upper edge of steel girder
y_{vl} : distance from centroid axis of composite girder to lower edge of steel girder
y_{vc} : distance from centroid axis of composite girder to concrete slab
d_0 : distance from centroid of steel girder to centroid of concrete slab
M_1: bending moment due to the first phase dead load which is resisted with steel girder only
M_2: bending moment due to the second phase dead load which is supported with the composite cross section.

When it suffers negative bending moment, as the concrete section is in tension, stresses are calculated by neglecting concrete and only considering reinforcement as shown in Fig.5.2.6. In this case, it is assumed that no slip occurs between the steel girder and the concrete slab, conforming to the Bernoulli Euler assumption.



a) Stress due to M_1 b) Stress due to M_2 c) Total stress

Fig.5.2.5 Elastic stress of live load composite girder subjected to positive bending moment



(a) Strain distribution (b) Stress distribution

Fig.5.2.6 Stress distribution of composite girder subjected to negative bending moment

5.2.4 Temperature change, shrinkage and creep of composite girders

(1) Derivation of analytical equations for temperature distribution specified by Specification for Road Bridges

Specification for Road Bridges, II: Steel Bridge Part2), specifies that temperature change of 10oC between the girder and the concrete slab as shown in Fig.5.2.7 for the design of composite girder.

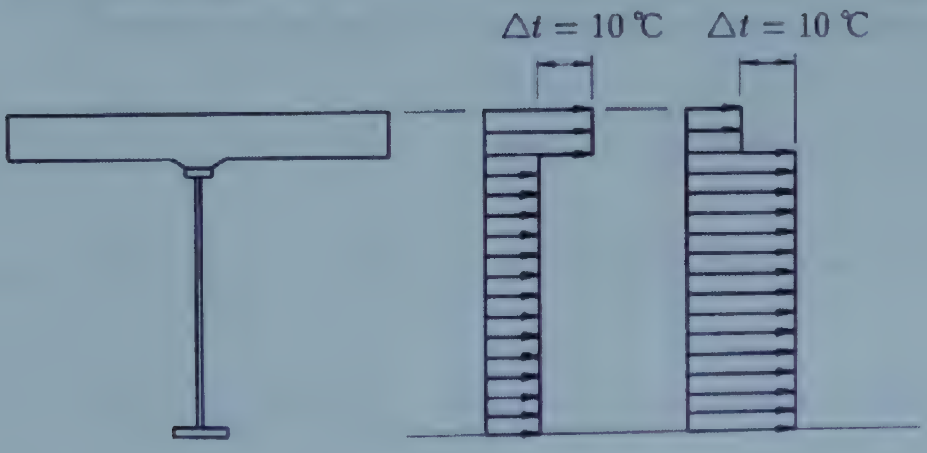
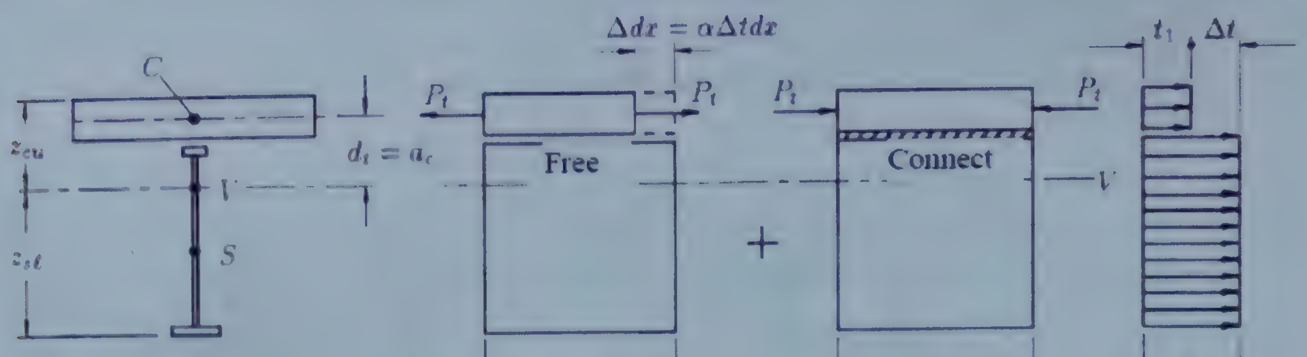
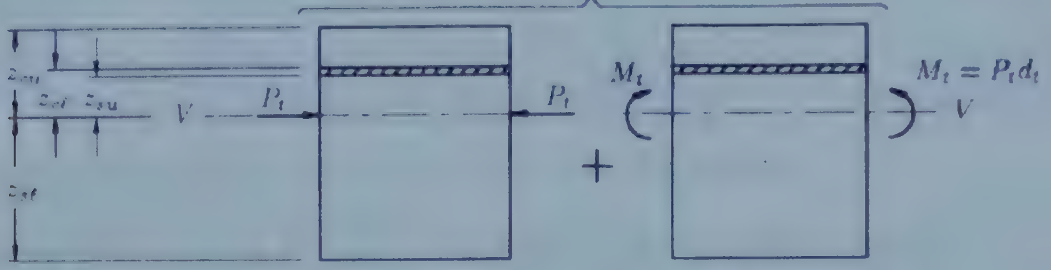


Fig.5.2.7 Temperature change used for design of composite girder



(a) Composite section (b) Free displacement (c) Pre-stress and combine (eccentric axial force) (d) Temperature distribution



(e) Axial force (f) Bending moment

Fig.5.2.8 Method to obtain sectional forces due to temperature change

Fig.5.2.8 shows the method to obtain sectional forces of composite bridges caused by temperature difference specified by Specification for Road Bridges. Assuming that temperature of steel girder t_2 is higher than that of concrete slab t_1, the temperature change is expressed by□□*t* = t_2 - t_1. In other words, if the steel temperature is set at the basis, concrete temperature drops by □*t*. As shown in Fig. 5.2.8 (b), let the composite action free on the girder with length of *dx*, the concrete slab moves freely due to the temperature change. Then, concrete shrinks by □*dx*. However, in reality, the steel girder and the concrete slab composite by shear connectors. Therefore, it is necessary to let the tensile force P_t act on the centroid of the concrete slab *C* to pull back to the original length *dx*. P_t can be obtained by Hooke's law.

$$P_t = \varepsilon_t E_c A_c = \alpha \Delta t E_c A_c \tag{5.2.10}$$

where, E_c, A_c: Young's modulus and cross-sectional area of concrete, α: linear expansion coefficient.

After the concrete slab pulled back to the initial length, the steel girder and the concrete slab composite and P_t is released, leading to the condition as shown in Fig.5.2.8(c). The compressive axial force P_t works with an eccentric distance of d_t. Therefore, the stress and deformation of composite girder can be obtained by superposing Fig.5.2.8 (b) and (c). Fig.5.2.8(c) can be separated into (e) and (f). In conclusion, stresses due to temperature change can be calculated by summing up (b), (e) and (f).

$$\left.\begin{array}{ll}
\text{Stress at upper edge of RC slab} & \sigma_{cu,t} = \dfrac{P_t}{nA_v} + \dfrac{M_t}{nI_v} z_{cu} - \alpha\Delta t E_c \\[2ex]
\text{Stress at lower edge of RC slab} & \sigma_{cl,t} = \dfrac{P_t}{nA_v} + \dfrac{M_t}{nI_v} z_{cl} - \alpha\Delta t E_c \\[2ex]
\text{Stress at upper edge of steel girder} & \sigma_{su,t} = \dfrac{P_t}{A_v} + \dfrac{M_t}{I_v} z_{su} \\[2ex]
\text{Stress at lower edge of steel girder} & : \sigma_{st,t} = \dfrac{P_t}{A_v} - \dfrac{M_t}{I_v} z_{st}
\end{array}\right\} \tag{5.2.11}$$

where, A_v, I_v: cross-sectional area and moment of inertia of composite girder, $n = E_s / E_c$: ratio of Young's of modulus of steel and concrete. The bending moment acting on the centroid of the composite girder can be calculated.

$$M_t = P_t \cdot d_t = \alpha \Delta t E_c A_c d_t \qquad (5.2.12)$$

Characteristic of the analysis considering temperature change is that stress is a function of not only strain but also temperature change in the relationship between stress and strain as shown in Eq.(4.4.1) and Eq.(4.4.2).

(2) Stress and deformation of composite girder due to shrinkage

A method to calculate stress and deformation of composite girder due to shrinkage is basically same as that due to temperature change. However, as shrinkage is accompanied by time dependent behavior, *Ec,s* shown in Eq.(5.2.14) should be used to calculate sectional properties for shrinkage analysis instead of *Ec* for temperature change analysis. The symbols in Fig.5.2.8 should be replaced asfollows.

$$V \rightarrow V_s, \quad d_t \rightarrow d_s, \quad \alpha\,\Delta t\,dx \rightarrow \varepsilon_{s,t}\,dx$$

Then, P_s and M_s can be calculated as follows (Eq. (5.2.10) and Eq. (5.2.12))

$$\left.\begin{aligned} P_s &= E_{c,s}\varepsilon_{s,\infty}A_c \\ M_s &= P_s d_s \end{aligned}\right\} \qquad (5.2.13)$$

where,

$$E_{c,s} = \frac{E_c}{1+\frac{1}{2}\varphi_{s,\infty}} \qquad (5.2.14)$$

Ratio of Young's modulus of steel to that of concrete is obtained as below.

$$n_s = n\left(1+\frac{1}{2}\varphi_{s,\infty}\right) \qquad (5.2.15)$$

This *ns* should be used to calculate the centroid of the composite section and section properties (Fig.5.2.9).

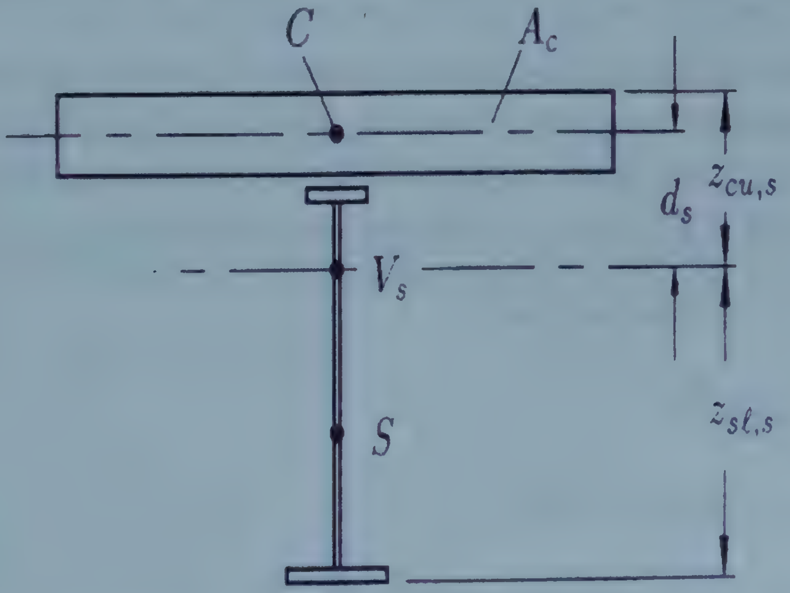


Fig.5.2.9 Symbols of centroid and distances in calculating stresses due to shrinkage

The change of stress at the upper edge of the concrete slab and at the lower edge of the steel girder due to shrinkage can be calculated using the following equations.

$$\left.\begin{array}{ll} \text{Upper edge of RC slab} & \Delta\sigma_{cu,s} = \dfrac{1}{n_s}\left(\dfrac{P_s}{A_{v,s}} + \dfrac{M_s}{I_{v,s}} z_{cu,s}\right) - E_{c,s}\varepsilon_{s,\infty} \\ \\ \text{Lower edge of steel girder} & \Delta\sigma_{st,s} = \dfrac{P_s}{A_{v,s}} - \dfrac{M_s}{I_{v,s}} z_{st,s} \end{array}\right\} \qquad (5.2.16)$$

The Specification for Road Bridges, II. Steel Bridge Part[2)], gives the following design values to evaluate stresses and deformations of the composite girder due to shrinkage.

$\varepsilon_{s,\square\square}\,20\square 10^{\square 5}$ and $\varphi_{s,\square\square\square}\,4.0$

This ultimate shrinkage is not the amount of concrete shrinkage itself but the value including the restrained effect due to reinforcement with 2% in the concrete slab.

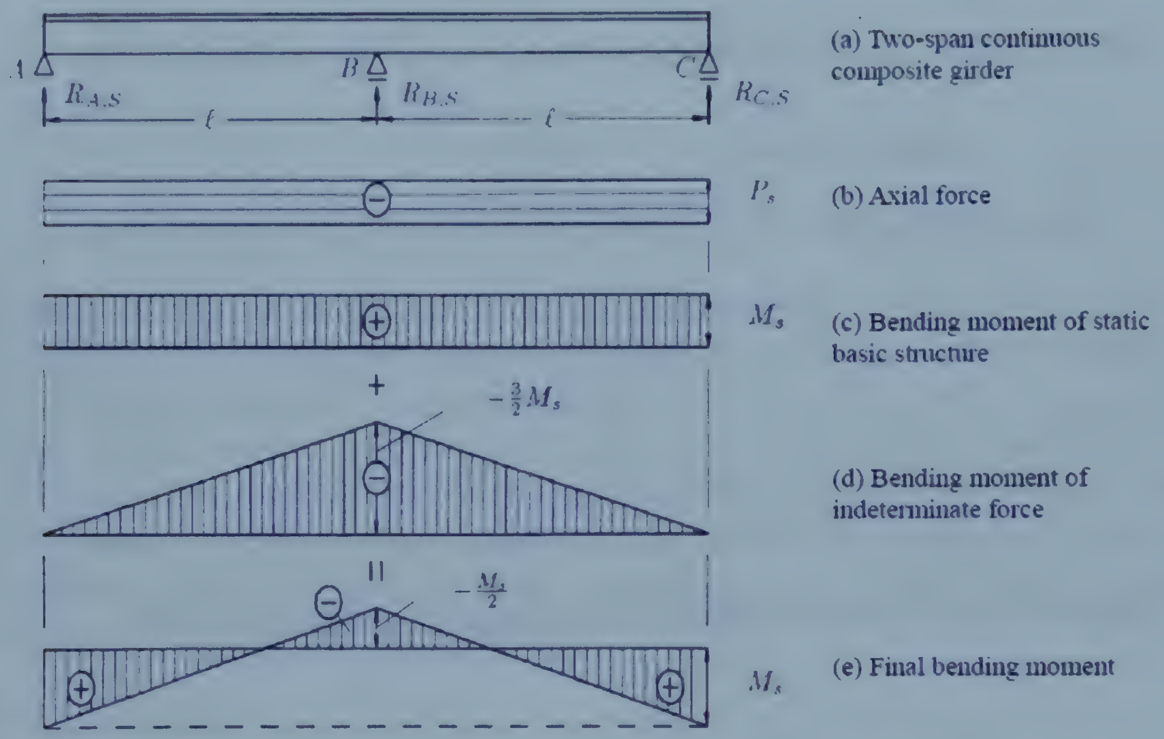


Fig.5.2.10 Sectional forces of two-span continuous composite girder due to shrinkage

(3) Sectional forces of two-continuous-span composite girder due to shrinkage

Taking a two-continuous-span composite girder (Fig.5.2.10) as an example, let us find sectional forces caused by shrinkage. As for axial forces, constant compressive forces P_S work along the full girder length, as shown in Fig. 5.2.10(b). Next, let the intermediate support free and consider the simply supported composite girder with span length of 2ℓ. Then, constant bending moments M_S due to shrinkage work along the full girder length, as shown in Fig. 5.2.10(c). An indeterminate force due to Ms, in other words, the reaction at an intermediate support can be obtained as follows.

$$R_{B,S} = \frac{3M_s}{\ell} \qquad (5.2.17)$$

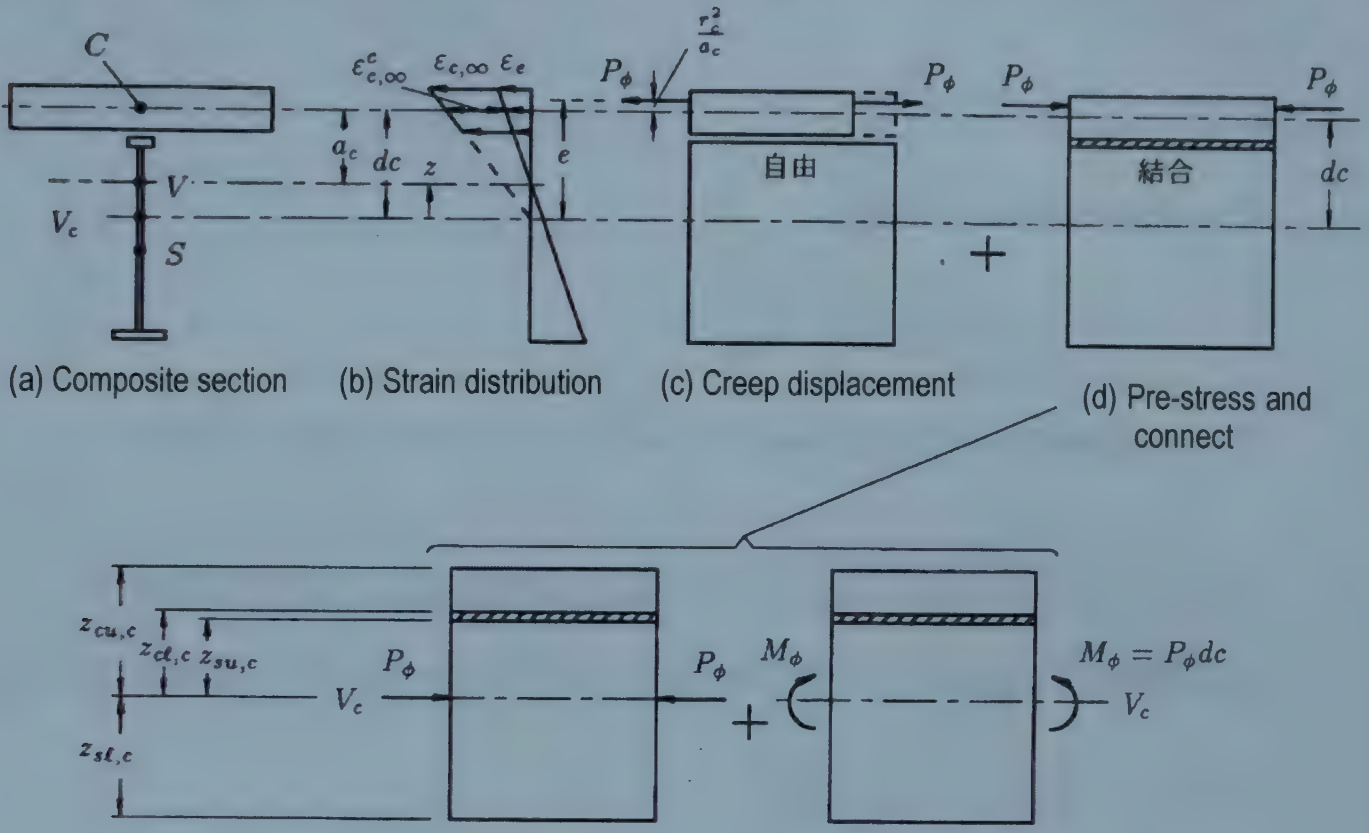
Then, $R_{B,S}$ are derived as below.

$$R_{A,S} = R_{C,S} = -\frac{3M_s}{2\ell} \qquad (5.2.18)$$

Then, the bending moment at the intermediate support caused by indeterminate force is obtained as below.

$$M'_{B,S} = -\frac{3M_s}{2} \qquad (5.2.19)$$

Its bending moment diagram is shown in Fig. 5.2.10(d). The final bending moment diagram becomes Fig. 5.2.10(e) by adding (c) and (d). There is the negative bending moment at the intermediate support, which is half of the positive bending moment. This negative bending moment may cause cracks in the concrete slab.



(a) Composite section (b) Strain distribution (c) Creep displacement (d) Pre-stress and connect

(e) Axial action (f) Bending moment action

Fig.5.2.11 Method to obtain sectional forces due to creep

(4) Stress of the composite girder due to creep

The analysis for creep is basically same as that for shrinkage. Fig.5.2.11 shows the method to obtain sectional forces of the composite girder due to creep. The basic axis for creep is expressed by *Vc*, which is obtained by *nc* instead of *n*.

$$n_c = n\left(1 + \frac{1}{2}\varphi_{c,\infty}\right) \tag{5.2.20}$$

Fig.5.2.11(b) shows the strain distribution when the composite girder takes constant bending moments.

The elastic strain at the upper edge of the concrete slab is expressed as ε_e. When the composite action is released between the steel girder and the concrete slab released, the strain $\varepsilon_{c,t}$ occurs due to creep.

Fig. 5.2.11(c) shows its deformation. The required force P to pull back is obtained as follows.

$$P_\varphi = E_{c,c}\int_{A_c} \varepsilon_{c,\infty} dA = E_{c,c} A_c \varepsilon^C_{c,\infty} \tag{5.2.21}$$

where,

$$E_{c,c} = E_c / \left(1 + \varphi_{c,\infty}/2\right)$$

$\varepsilon^C_{c,\infty}$: Final value of free creep strain at centroid C of the RC slab

After getting the slab back to the original position by applying the tensile force P, make the concrete slab and the steel girder composite and release P, eccentric axial compression P works, as shown in (d). Therefore, stress change due to creep can be obtained by superposing (c) and (d). It is understood more easily if (d) is dealt with (e) and (f) separately.

$$\varepsilon^{C}_{c,\infty} = \frac{\sigma_{c,0}}{E_c}\varphi_{c,\infty} = \frac{N_c'}{E_c A_c}\varphi_{c,\infty} \tag{5.2.22}$$

Where,

$$N_c' = \frac{M_d}{nI_v}a_c A_c \tag{5.2.23}$$

Then P_φ is obtained as follows.

$$P_\varphi = E_{c,c}A_c\frac{N_c'}{E_c A_c}\varphi_{c,\infty} = \frac{2\varphi_{c,\infty}}{2+\varphi_{c,\infty}}N_c' \tag{5.2.24}$$

The acting position of P_φ is expressed by the Eq. (5.2.25) referring Fig.5.2.11(b).

$$P_{\varphi,e}\cdot e = E_{c,c}\int_{A_c}\varepsilon^{z}_{c,\infty}dA\cdot z = P_\varphi\left(d_c+\frac{r_c^2}{a_c}\right) \tag{5.2.25}$$

The position is higher by r_c^2/a_c from the center line of the concrete slab. Therefore, M_φ becomes as below.

$$M_\varphi = P_\varphi\cdot e = P_\varphi\left(d_c+\frac{r_c^2}{a_c}\right) \tag{5.2.26}$$

where,

r_c: Radius of section modulus of concrete ($r_c^2=I_c/A_c$)

The change of stress due to creep can be calculated by the following equations.

$$\left.\begin{aligned} &\text{Upper edge of RC slab} && \Delta\sigma_{cu,c} = \frac{1}{n_c}\left(\frac{P_\varphi}{A_{v,c}}+\frac{M_\varphi}{I_{v,c}}z_{cu,c}\right)-\sigma_{c,0}\frac{2\varphi_{c,\infty}}{2+\varphi_{c,\infty}} \\ &\text{Lower edge of steel girder} && \Delta\sigma_{sl,c}=\frac{P_\varphi}{A_{v,c}}-\frac{M_\varphi}{I_{v,c}}z_{sl,c} \end{aligned}\right\} \tag{5.2.27}$$

where, $A_{v,c}$ and $I_{v,c}$: cross-sectional area and moment of inertia using n_c.

5.2.5 Bending strength of Composite girders

(1) Introduction

The bending strength of composite girders can be obtained depending on the required performances such as that local buckling does not occur, local buckling occurs, or elastic buckling occurs, in other words, on types of failure of the section. Overseas design specifications (AASHTO, Eurocode) defines the section class to satisfy the above required performances.

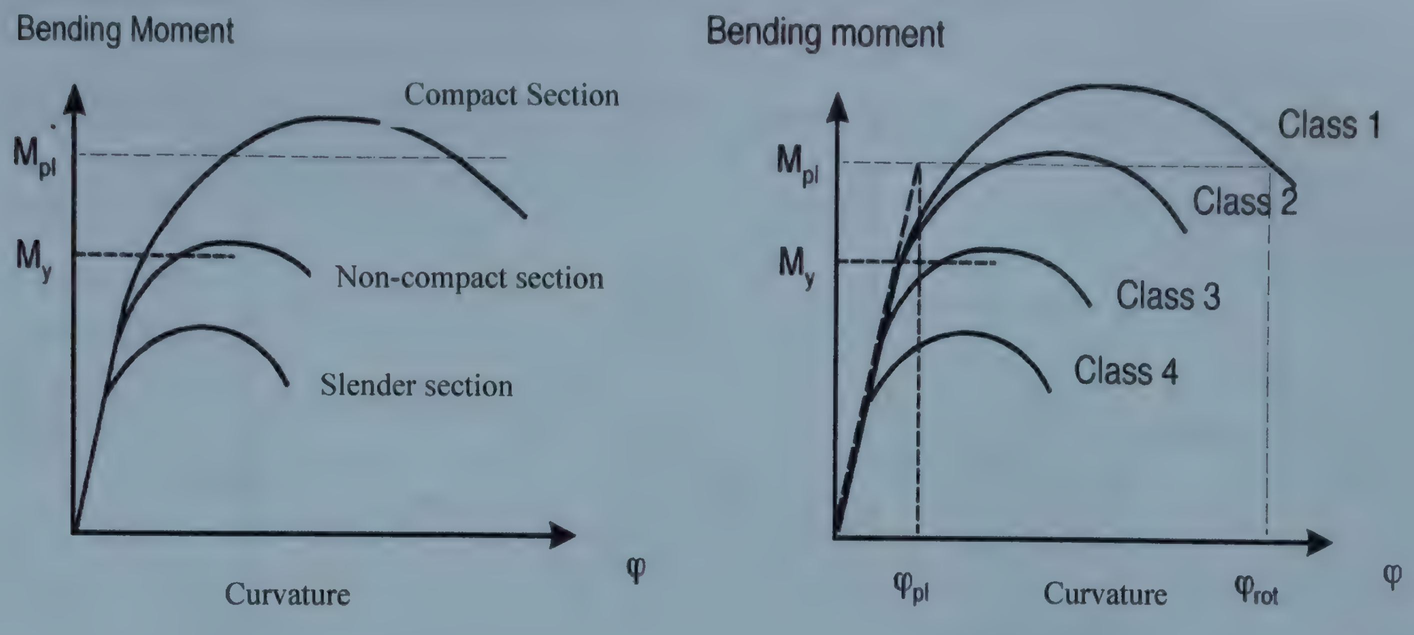
a) AASHTO LRFD (see Fig.5.2.12(a)) 3)

Compact section: Maximum bending moment of a section reaches the plastic moment.

Non-compact section: Part of the steel section is yield, maximum bending moment of a section cannot reach the plastic moment due to local buckling.

Slender section: Maximum bending moment of a section cannot reach the yield moment.

The plastic moment is the moment when all steel members reach the yield stress and concrete reaches 85% of the compressive stress as shown in Fig.5.2.13. The yield moment is the moment when either upper or lower edge reaches the yield stress, assuming an elastic condition.



(a) AASHTO (b) EUROCODE

Fig.5.2.12 Relation of bending moment vs. rotation angle of section classes

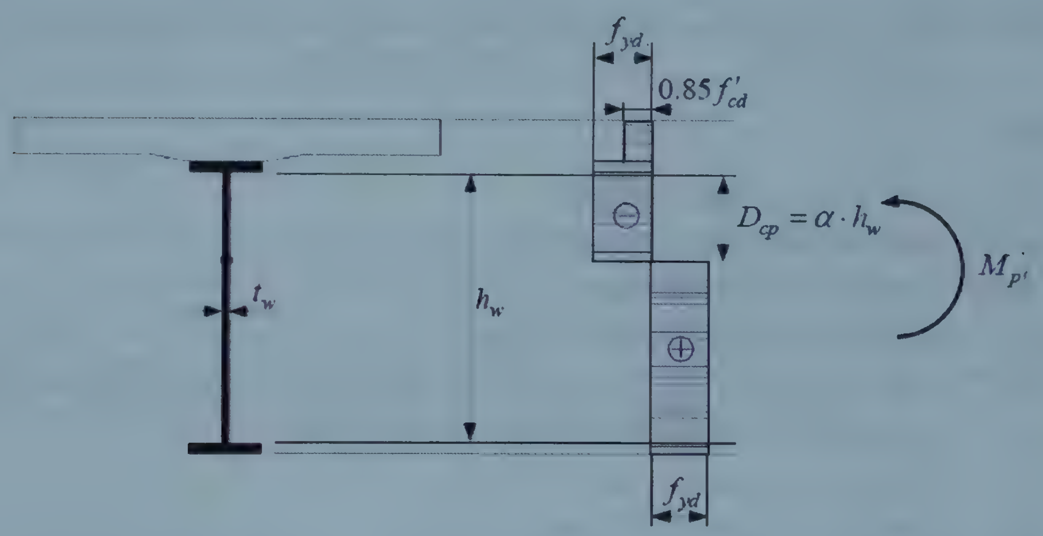


Fig.5.2.13 Stress distribution of compact section at full plastic bending moment

b) Eurocode (see Fig.5.2.12(b))4)

Class 1: A composite section with a good rotational capability which re-distribute the moment after the formation of a plastic hinge in a statically indeterminate structure

Class 2: A composite section, which can reach the full plastic moment state, with a limited rotational capability due to local buckling of steel and/or concrete compression failure

Class 3: A composite section in which the compression flange reaches yield point but moment cannot reach to the level of full plastic moment because of local buckling

Class 4: A composite section which is slender and reaches failure due to local buckling without yielding of compression flange.

In plastic design, after a plastic hinge is formed, it is assumed that bending moment is re-distributed in a structure. Class 1 requires that the strength must not be less than the plastic moment until a rotation

Angle φ_{rot} so that redistribution can occur. There is no requirement about plastic rotation performance for the compact section AASHTO LRFD. Therefore, the compact section of AASHTO LRFD corresponds to Class 2 of Eurocode, non-compact section to Class 3 and slender section to Class 4, respectively.

Table 5.2.2 Section class in Standard Specification of Hybrid Structures

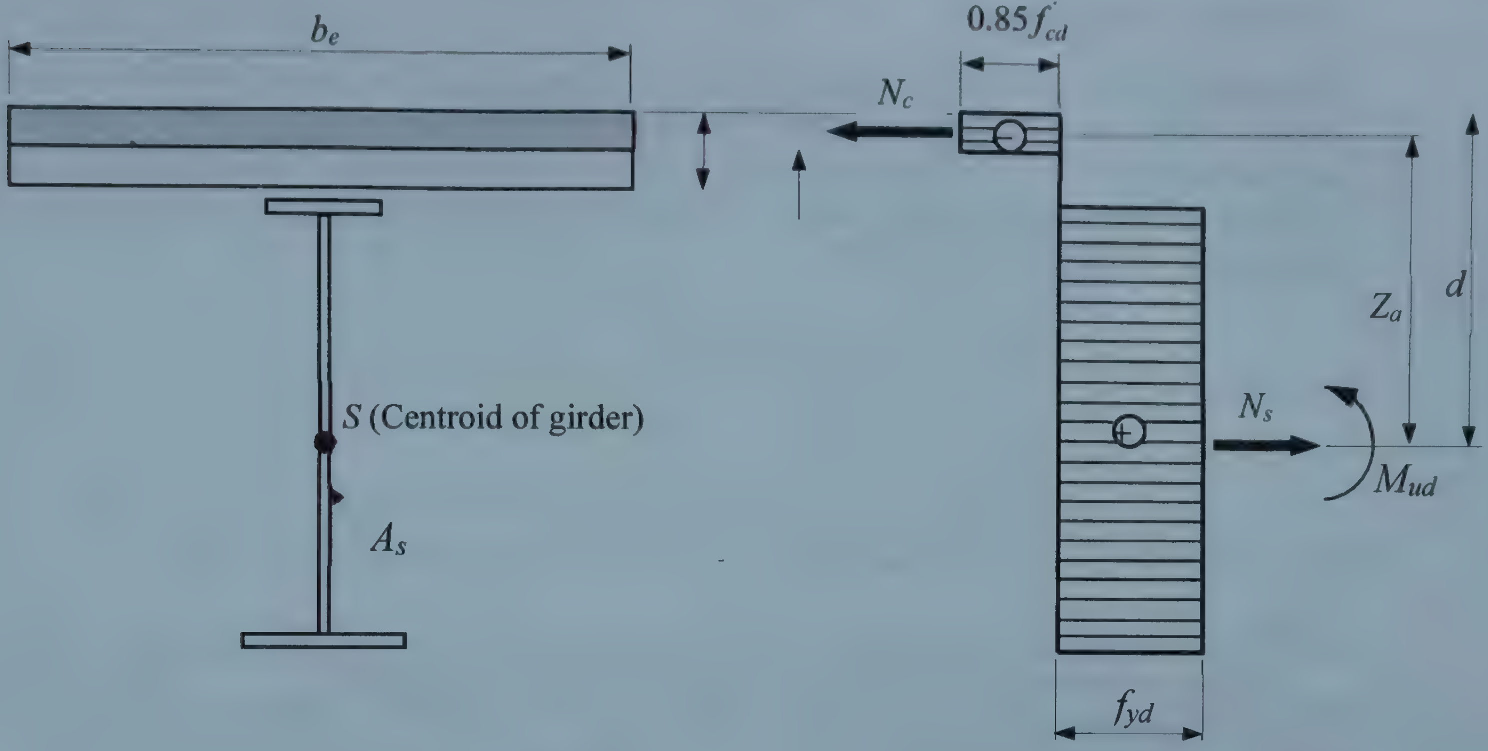
	Positive bending moment	Negative bending moment
Compact section	$\frac{h_w}{t_w} \leq \frac{2.0}{\alpha}\sqrt{\frac{E_s}{f_{yd}}}$ (where, $\alpha < 0.4$)	Web $\frac{h_w}{t_w} \leq 3.8K_1\sqrt{\frac{E_s}{f_{yd}}}$ $\left(K_1 = 1 - 0.63N'_d/N'_{pd}\right)$ Compressive flange $\frac{b_f}{2t_f} \leq 0.37\sqrt{\frac{E_s}{f_{yd}}}$
Non-compact section	$\frac{h_w}{t_w} \leq \frac{1.7\Lambda}{0.67 + 0.33\psi}\sqrt{\frac{E_s}{f_{yd}}}$ $(\psi > -1.0)$ $\frac{h_w}{t_w} \leq 2.5\Lambda(1-\psi)\sqrt{-\psi}\sqrt{\frac{E_s}{f_{yd}}}$ $(\psi \leq -1.0)$: factor of influence of initial moment $\Lambda = \left\{1 - 0.1\left(\frac{M_{d1d}}{M_{ysd}}\right) + 2.31\left(\frac{M_{d1d}}{M_{ysd}}\right)^2\right\}$ $\left(\frac{M_{d1d}}{M_{ysd}} \geq 0.4\right)$	Web Ratio of width to thickness of compressive flange $\frac{b_f}{2t_f} \leq 0.45\sqrt{\frac{E_s}{f_{yd}}}$
Slender section	Others except above	Others except above

where,

hw : web height of steel member
tw : web thickness of steel member
bf : width of compressive flange
tf : thickness of compressive flange
α : ratio of distance from plastic neutral axis to top of compressive web over web height
ψ : parameter showing inclined rate of stress in web
Es : Young's modulus of steel
fyd : design yield strength of steel
N'd : design compressive force
N'pd : design full yield axial yield compression (N'_{PD} ▯ $A_S f_{yd}$ / ▯$_b$)
Mysd : design yield bending moment of steel girder
Md1d : design bending moment of pre-composite dead load
As : cross-sectional area of steel member
γb : member factor

(2) Section Groups

Table 5.2.2 shows the section groups defined by Standard specification for hybrid structures (2014)[5)]. In Eurocode and AASHTO, the limit values of section groups of the composite bridge subjected to positive bending moment is assumed to be the same as that of the steel girder. Standard Specification for Hybrid Structures decides section groups based on the elastic and plastic FE analysis by Gupta. When the composite girder is subjected to positive bending moment, local buckling does not occur in the upper flange which is fixed to the concrete slab due to shear connectors. Therefore, as local buckling may occur only in the web, section group is determined by the ratio of width to thickness of webs and compressive area.



(a) Cross-section (b) Stress distribution

Fig.5.2.14 Stress distribution at full plastic bending moment subjected to positive bending moment

(3) Compact Section

a) Positive bending moment

The plastic neutral axis of the composite girder subjected to positive bending moment is located in the concrete slab in most cases. Let us calculate the position of plastic neutral axis and design bending moment by referring the stress distribution as shown in Fig.5.2.14. If the plastic neutral axis is located in the steel girder, the position of plastic neutral axis and design bending moment can be found by the equilibrium condition of compressive forces of concrete and steel and tensile force of steel.

$$x = \frac{A_s f_{yd}}{0.85 f'_{cd} b_e} \leq h_c \tag{5.2.28}$$

$$Z_a = d - \frac{1}{2}x \tag{5.2.29}$$

$$M_{ud} = \frac{A_s f_{yd} Z_a}{\gamma_b} \tag{5.2.30}$$

where,

x: distance between upper edge of concrete and plastic neutral axis
h_c: thickness of concrete slab
A_s: cross-sectional area of steel girder
f_{yd}: design yield strength of steel
f'_{cd}: design compressive strength of concrete
b_e: effective width of concrete section in compression
Z_a: arm length
d: distance between upper edge of concrete and centroid of steel member
M_{ud}: design bending moment
γ_b: structural member factor

b) Negative bending moment

The plastic neutral axis of the composite girder subjected to negative bending moment is located in the steel girder in most cases as shown in Fig.5.2.15. The position of plastic neutral axis and design bending moment are calculated by the plastic stress condition of the steel section and only the longitudinal reinforcement in the concrete slab neglecting concrete section. The position of plastic neutral axis and design bending moment is given by Eq. (5.2.31) and Eq. (5.2.32).

$$x' = h_c + t_f + \frac{f_{yd} A_s - 2 f_{yd} A_{fu} - f_{ryd} A_r}{2 f_{yd} t_w} \tag{5.2.31}$$

$$M_{ud} = \frac{\left\{ f_{yd} A_s (d - d') - f_{yd} A_{fu} (2h_c - 2d' + t_f) - f_{yd} t_w (x' - h_c - t_f)(x' + h_c + t_f - 2d') \right\}}{\gamma_b} \tag{5.2.32}$$

where,

x': distance between upper edge of concrete and plastic neutral axis
h_c: thickness of concrete slab
t_f: thickness of compressive flange
f_{yd}: design yield strength of steel
A_s: cross-sectional area of steel member
f_{ryd}: design yield strength of longitudinal reinforcement
A_{fu}: cross-sectional area of upper flange
A_r: cross-sectional area of longitudinal reinforcement
t_w: thickness of web
M_{ud}: design bending moment
d: distance between upper edge of concrete and centroid of steel girder
d': distance between upper edge of concrete and reinforcement
$\square_b$: structural member factor

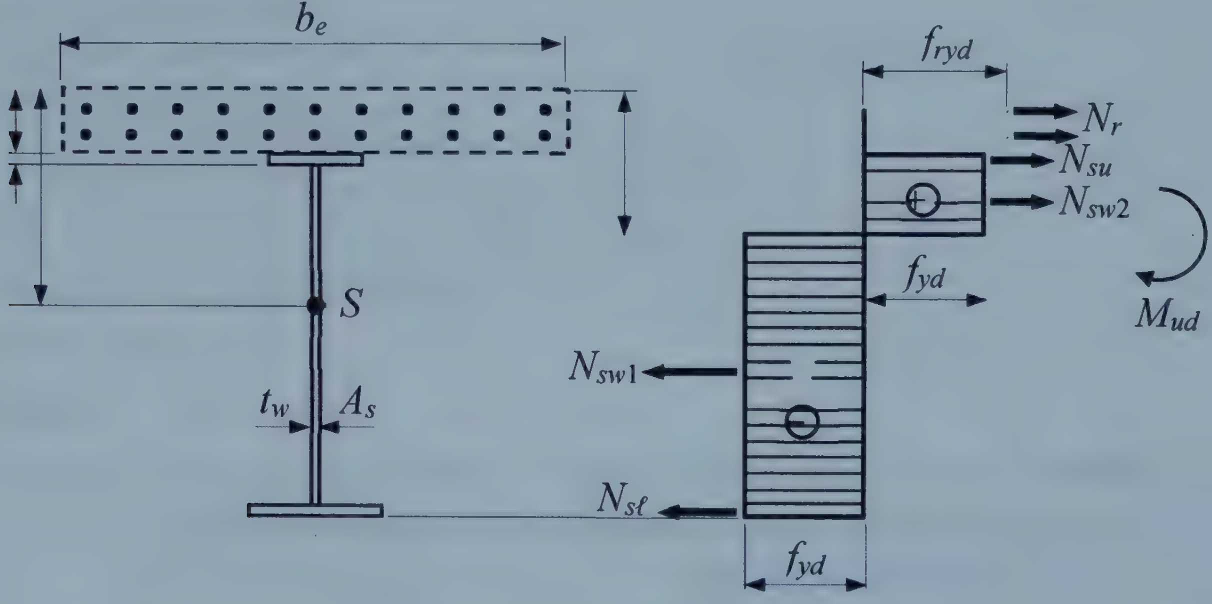


Fig.5.2.15 Stress distribution at full plastic bending moment subjected to negative bending moment

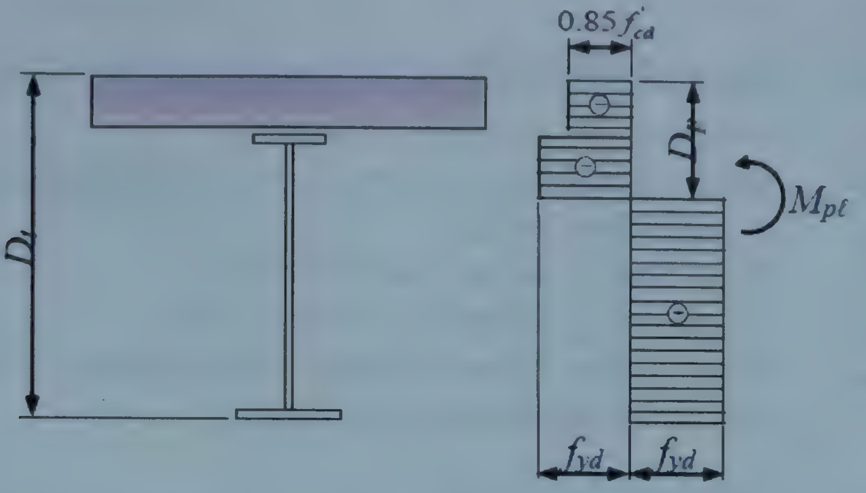


Fig.5.2.16 Stress distribution and plastic neutral axis at full plastic bending moment

c) Considering the effect of bending failure of concrete slab subjected to positive bending moment

The bending strength is obtained from Eq. (5.2.30) under the condition that no local buckling occurs and compressive failure of concrete is neglected. In this section, the same approach is used to calculate the bending strength, although the effect compressive failure of concrete is included.

When no local buckling occurs and compressive failure of concrete is neglected, the bending strength can reach the plastic moment M_{pl}. In the plastic moment, concrete above the plastic neutral axis conforms to 85% of the compressive strength of concrete block and steel conform to the design strength (Fig.5.2.16).

When compressive failure of concrete is considered, the design bending moment can be calculated by Eq. (5.2.33)

$$M_{ud} = \begin{cases} \dfrac{M_{pl}}{\gamma_b} & \left(\dfrac{D_p}{D_t} \leq 0.15\right) \\ \left(1.05 - 0.33\dfrac{D_p}{D_t}\right)\dfrac{M_{pl}}{\gamma_b} & \left(0.15 < \dfrac{D_p}{D_t} < 0.4\right) \end{cases} \qquad (5.2.33)$$

M_{ud}: Design plastic bending moment
M_{pl}: Plastic Moment
D_p: Distance between upper edge of concrete and the plastic neutral axis
D_t: Total depth of the composite section (Fig.5.2.16)
□$_b$: Structural member factor

In Eq. (5.2.33), the bending strength is reduced due to the effect of compressive failure of concrete for $0.15 < D_p/D_t$.

(4) Non-compact section

For non-compact section, part of steel member is yield but it fails due to elastic buckling. The effect of pre-composite action is considered as the initial stress condition for the post-composite behaviours, and then it is verified that the tensile and compressive edges do not reach the limits of tensile and compressive strength.

a) Composite girder for live load

The verification equation for composite girder for dead and live loads is shown in Eq. (5.2.34). The effect of creep of concrete caused by the post-composite dead loads such as pavements should be considered, however, change of stress due to this effect is small and neglected in this equation.

$$\gamma_i\left(\frac{M_{d1d}}{M_{sud}} + \frac{M_{d2d} + M_{ld}}{M_{vud}} + \frac{M_{shnd}}{M_{vuds}}\right) \leq 1.0 \qquad (5.2.34)$$

where,

M_{d1d}: Design bending moment due to the first phase dead load which is resisted with steel girder only

M_{sud}: Design bending moment of steel section

M_{d2d}: Design bending moment due to the second phase dead load which is supported with the composite cross section

M_{ld}: Design bending moment due to live load

M_{vud}: Design bending strength of composite section

M_{shnd}: Design bending moment considering axial forces due to shrinkage of concrete
This moment expressed by Eq. (5.2.35).

$$M_{shnd} = M_{shd} + \frac{N_{shd}}{A_{v2}} W_{vs} \qquad (5.2.35)$$

M_{shd}, N_{shd}: Design moment and axial force due to shrinkage of concrete

W_{vs}: Section modulus of composite section applying for the ratio of Young's modulus considering shrinkage of concrete obtained from Eq.(5.2.36)

$$n_{s.\varphi} = n\left(1 + \varphi_{s.\varphi}/2\right) \qquad (5.2.36)$$

n: Ratio of Young's modulus

$\varphi_{s,\varphi}$: Creep coefficient accompanied by shrinkage of concrete (=usually 4.0), depending on concrete mixture and environmental conditions

A_{v2}: Cross sectional area of equivalent composite section with concrete section replaced by steel applying for the ratio of Young's modulus considering shrinkage of concrete obtained from Eq.(5.2.36)

M_{vuds}: Design bending strength of equivalent composite section with concrete section replaced by steel applying for the ratio of Young's modulus considering shrinkage of concrete obtained from Eq.(5.2.36)

γ_i: Structural factor

Bending strength is obtained by conforming to the Bernoulli-Euler assumption and the linear relationship between stress and strain of steel and concrete and by considering effective width of the concrete slab and the steel section, respectively. The limit of material strength is as follows.

1) The limit value of concrete in compression is design compressive strength f'_{cd}. Concrete in tension is generally neglected.
2) The limit value of structural steel in tension or compression is design yield strength f_{yd} or f'_{yd}.
3) The limit value of reinforcement in tension or compression is design yield strength f_{syd} or f'_{syd}. However, the reinforcement inside concrete slab in compression is generally neglected.

b) Composite girder for dead and live loads

In composite girder for dead and live loads, the composite section resists both dead and live loads. Neglecting the first term of Eq. (5.2.34) and adding the first phase dead load to the second term, the verification equation is conducted by Eq.(5.2.37).

$$\gamma_i\left(\frac{M_{dd}+M_{ld}}{M_{vud}} + \frac{M_{shnd}}{M_{vuds}}\right) \leq 1.0 \qquad (5.2.37)$$

where,

M_{dd}: design bending moment due to dead loads

5.2.6 Shear strength of composite girders

The design shear strength of the steel web V_{wd} is calculated by the following Basler equation.

$$\frac{V_{wd}}{V_{sy}}=\frac{1}{\gamma_b}\left(\frac{f_{vcr}}{f_{vy}}+\frac{\sqrt{3}}{2}\frac{1-f_{vcr}/f_{vy}}{\sqrt{1+\alpha^2}}\right) \tag{5.2.38}$$

$$V_{sy}=f_{vy}h_w t_w \tag{5.2.39}$$

where,

V_{wd}: Design shear strength of steel web.
V_{sy}: Shear yield Strength.
F_{vcr}: Shear buckling strength of web
F_{vy}: Shear yield strength
a: Aspect ratio
h_w: Web height of steel member
t_w: Thickness of web of steel member
$\square_b$: structural member factor

The shear buckling strength of a web is calculated by the following equations.

$$f_{vcr}=\begin{cases} f_e & (f_e \le 0.8 f_{vy}) \\ \sqrt{0.8 f_e f_{vy}} & (f_e \ge 0.8 f_{vy}) \end{cases} \tag{5.2.40}$$

$$f_e=k_s\frac{\pi^2 E_s}{12(1-\nu^2)}\left(\frac{t_w}{h_w}\right)^2 \tag{5.2.41}$$

where k_s is calculated by the following equation.

$$k_s=\begin{cases} 5.34+4.00/\alpha^2 & (\alpha \ge 1.0) \\ 4.00+5.34/\alpha^2 & (\alpha \le 1.0) \end{cases} \tag{5.2.42}$$

where,

f_{vcr}: Shear buckling strength
f_{vy}: Shear yield strength
f_e: Elastic shear buckling strength
Es: Young's modulus of steel
$\square$: Poisson ratio
tw: Web thickness of steel member
hw: Web height of steel member
ks: Buckling factor for shear stress
α: Ratio of height from plastic neutral axis of web to compression area to web height.

The first term of the right-hand side of Eq. (5.2.38) is shear buckling strength and the second term is the post-buckling strength due to diagonal tension field. At the end of girder, the diagonal tension field may not be expected and, therefore, rigid vertical stiffeners are needed.

When the horizontal stiffeners are provided to the web, the shear strength can be obtained by applying Eq. (5.2.38) to the upper and lower panels and adding them.

Eq. (5.2.38) is the equation targeting only the steel member. The composite girder can be expected for higher shear strength, but it's not fully clarified. Therefore, Eq. (5.2.38) is used for composite members.

5.2.7 Verification of shear connectors of composite girders

Shear connectors are set at the connecting part between upper flange of steel members and concrete, and must satisfy Eq. (5.2.43) against the longitudinal shear force working at the connection part. As re-distribution of longitudinal shear forces is expected due to plastic deformation of shear connectors, shear connectors should have sufficient deformation performance.

$$\gamma_i \frac{q_{sd}}{q_{rd}} \le \alpha \quad \text{and} \quad \gamma_i \frac{V_{sd}}{V_{rd}} \le 1.0 \tag{5.2.43}$$

where,

□: Coefficient to consider effect of non-elasticity of shear connector
q_{sd}: Horizontal design shear force per unit length
q_{rd}: Horizontal design shear strength per unit length
V_{sd}: Total horizontal design shear force over a longitudinal length where the size, type and arrangement of shear connectors are kept same.
V_{rd}: Total horizontal design shear strength of the same length
□$_i$: structure factor

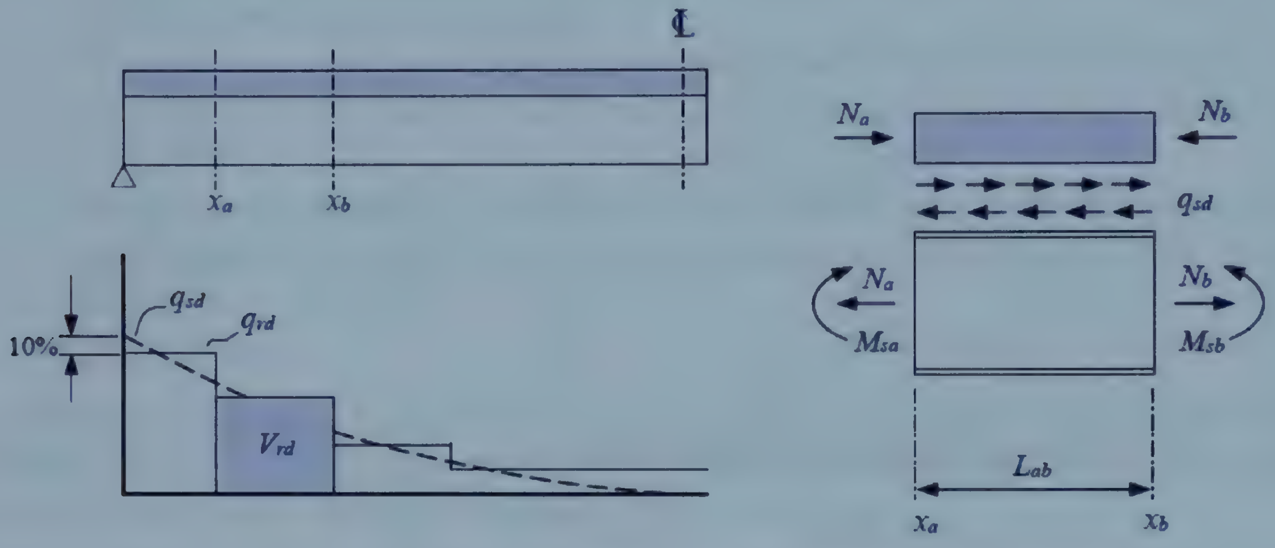


Fig.5.2.17 Relation of design lateral shear force and design strength

Non-elastic deformation of shear connectors is allowed at the ultimate state. Therefore, the first equation of Eq. (5.2.43) allows that acting lateral shear force q_{rd} exceeds design strength q_{rd} by 10% as shown in Fig.5.2.17. Although, the slip deformation is locally allowed in this case, it is known that the strength as a composite girder is secured, if the total shear force in the length (for example, L_{ab}) with the same type and arrangement of shear connectors is less than the total shear strength of shear connectors.

The relationship of total design shear forces, axial forces of the slab and acting lateral shear forces is expressed as follows.

$$V_{sd} = N_b - N_a = \int_{x_a}^{x_b} q_{sd} dx \tag{5.2.44}$$

where,

V_{sd}: Total horizontal design shear force over a longitudinal length where the size, type and arrangement of shear connectors are kept same.
N: Axial forces
q_{sd}: Horizontal design shear force per unit length.

On the other hand, total design strength can be obtained by the number of shear connectors and in the area Lab with the same type and arrangement of shear connectors and design strength V_{sud} of a shear connector, as follows.

$$V_{rd} = n_{ab} \cdot V_{sud} \qquad (5.2.45)$$

where,

V_{rd}: Total horizontal design shear strength of the same length.
n_{ab}: Number of shear connectors.
V_{sud}: Design Strength of a shear connector.

When is over 1.0 in the safety verification of Eq. (5.2.43), the plastic deformation of a shear connector is permitted at the ultimate state. Therefore, shear connectors should have sufficient ductility to guarantee the strength as a composite girder. Eurocode 4 specifies that a ductile shear connector must hold strength when the slip displacement of 6mm occurs. In general, the headed studs are classified as ductile connector. When ductile shear connectors are used, = 1.1.

5.3 Composite slab

5.3.1 Composite slab with steel plate

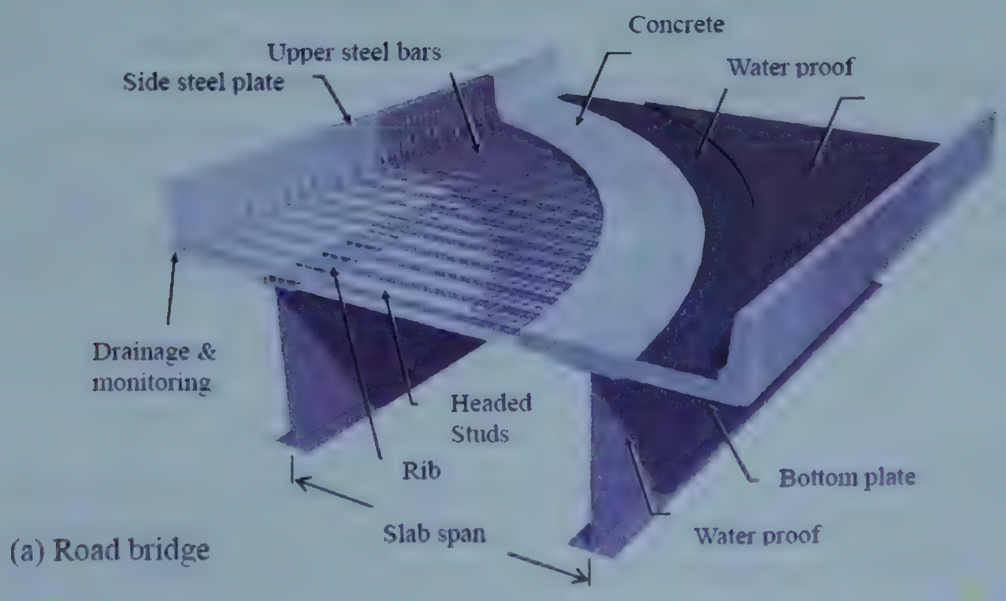
(1) Definition

In the composite slab with steel plate, the steel plate works as a formwork and a structural member and is composited with concrete by shear connectors. When it is fabricated in the vertical plane, a temporary formwork is placed on the opposite side of the steel plate. This steel plate has functions as a formwork and a structural member. When it is used as a bottom formwork, it is called bottom plate. Stiffeners are steel plates which stiffen steel plates before composited with concrete and work as shear connectors and structural member after completion and in service. Fig.5.3.1 shows two examples of composite slab with steel plate: the slab for road bridges and the wall structure of hybrid caisson.

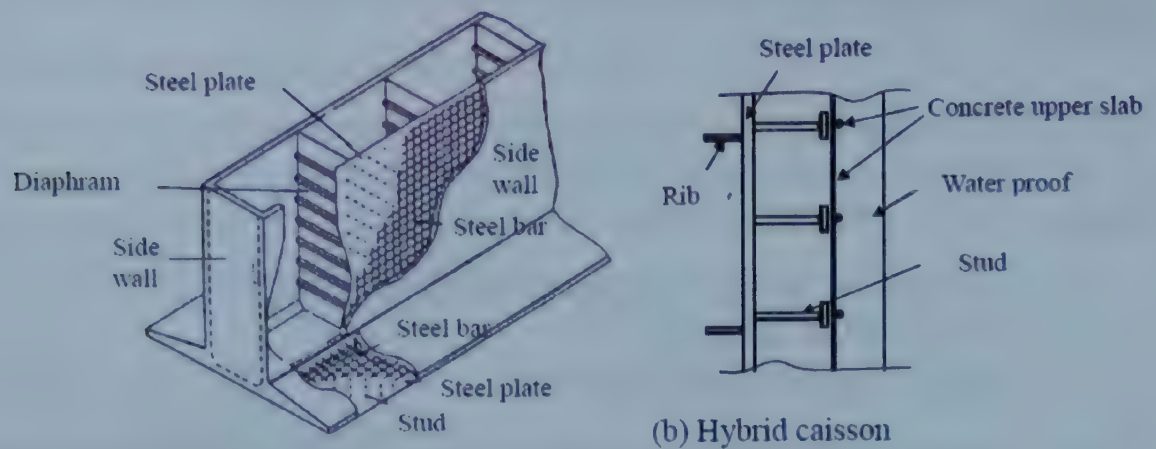
(2) Characteristics

The composite slab with steel plate combining steel plates and concrete have been widely used as bridge slabs, submerged tunnels, caissons, harbor structures such as floating jetty, rock sheds and so on. The steel plates or rolled steel sections or their combination can be a main member of this composite plate. In composite slab with steel plate, the shear connectors are attached to the steel plates which work as a formwork and a structural member, filled concrete and they are finally composited with concrete. When a structure consists of plural composite plates such as a submerged tunnel, another system can be used such that the composite slab with steel plate are used at the upper side and the steel concrete sandwich slab at the side and bottom, respectively.

(a) Road bridge



(b) Hybrid caisson



(b) Hybrid caisson

Fig.5.3.1 Example of steel plate concrete composite plate

There are many types of shear connectors used for the composite slab with steel plate such as headed studs, ribs, steel truss type and so on for road bridge slabs.

(3) Structural analysis

In design of the composite slab with steel plate, the same actions as plate of steel structures or concrete structures are generally considered. Both actions working only on the steel section at any stage under construction and working on the composite section after steel and concrete are composited must be evaluated. Therefore, the resistance of each structural element such as bottom plate, rolled steel section, reinforcement and concrete must be appropriately evaluated depending on under construction and in service period.

As for structural analysis to obtain responses of the composite slab with steel plate, thin-plate theory or beam theory is used depending on actions or characteristics of structures. The effect of nonlinear structural analysis is in principle considered, however, it can be treated as linear if its effect to design response is negligible.

(4) Strength

Strength of the composite slab with steel plate must be obtained both before steel and concrete are unified and after unified. Before steel and concrete are unified, only the steel plate is considered. After steel and concrete are unified, the composite plate is considered. As failure types of composite slab with steel plate, there are bending failure, shear failure, local push-out shear failure and so on. Shear failure after composite is described below.

When a composite plate subjected to out-of-plane shear forces can be assumed as a beam to resist shear forces as a one-way slab, it is verified against shear forces as a bar element. The design shear strength is expressed by Eq. (5.3.1).

Shear failure of the composite slab with steel plate due to out-of-plane shear forces is almost the same as the usual reinforced concrete members. However, as the shear strength may significantly reduce depending on types of shear connectors and arrangements, design shear strength of reinforced concrete cannot be applied. Therefore, the reduction factor α_1 is introduced to the ratio of tensile steel against the shear strength shared by concrete. In this method, the shear strength of concrete is taken as the design shear strength neglecting the reinforcing effect of shear connectors to evaluate it in a safety side. The same reduction factor provides the design strength in a safety side for composite plates with headed studs, PBLs and other shear connectors. However, when a space of shear connectors is longer than the usual one or block shear connectors are applied, the shear strength must be confirmed by experiments or other methods.

$$V_{cd} = \frac{\beta_d \cdot \beta_p \cdot f_{vcd} \cdot b \cdot d}{\gamma_b} \tag{5.3.1}$$

Where,

$f_{vcd} = 0.20\,(f'_{cd})^{1/3}$ (N/mm^2)
$\beta_d = (1000/d)^{1/4}$, whereas 1.5 for $\beta_d > 1.5$
$\beta_p = (100\alpha_1 \cdot p)1/3$, whereas 1.5 for $\beta_p > 1.5$

b: Unit width or width of the verified section (mm)
d: Effective depth (mm)
β_1: Reduction factor due to types and arrangements of shear connectors
p: Ratio of steel in tension
f_{vcd}: Design compressive strength of concrete (N/mm^2)
γ_b: Member factor (generally 1.3)

Next, at around the concentrated load or near the supports, out-of-plane shear forces are assumed to be resisted by a two-direction plate with a shape of cone or pyramid. The design strength is shown in Eq. (5.3.2) and assumed push-out shear failure type is shown in Fig.5.3.2.

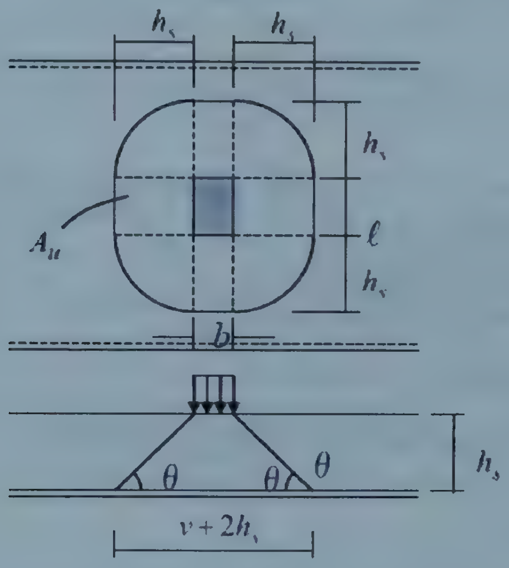


Fig.5.3.2 Assumed push-out shear failure type

Comparing the composite slab with steel plate with reinforced concrete members, although the tensile reinforcement is thin steel plates and the method of bond between steel and concrete is different, its push-out shear failure property due to out-of-plane shear forces is similar to that of reinforced concrete member. However, when the strength of shear connectors is insufficient, the local bond shear slip occurs and the design strength of the steel plate concrete composite plates is lower than that calculated for reinforced concrete members. Then, reduction factor β_2 is introduced to ratio of tensile steel.

$$V_{pcd} = \frac{\beta_d \cdot \beta_p \cdot \beta_r \cdot f_{pcd} \cdot u_p \cdot d}{\gamma_b} \qquad (5.3.2)$$

where,

$f_{vcd} = 0.20\ (f'_{cd})^{1/3}$ (N/mm^2), whereas $f_{pcd} \leq 1.2$ N/mm^2
$\beta_d = (1000/d)^{1/4}$, whereas 1.5 for $\beta_d > 1.5$
$\beta_p = (100\beta_2 \cdot p)1/3$, whereas 1.5 for $\beta_p > 1.5$
f_{vcd}: Design compressive strength of concrete (N/mm^2)
u: Circumference of loading plane
u_p : Circumference at verifying position, $d/2$ away from the loaded plane
d: Effective depth (mm)
β_2: Reduction factor due to types and arrangements of shear connectors
p: Ratio of steel in tension
γ_b: Member factor (generally 1.3)

5.3.2 Steel and concrete sandwich composite plates

(1) Definition

The steel and concrete sandwich composite plate is a composite structure consisting of two steel plates and concrete filled between two plates and they behave as a one piece. Concrete is poured between two steel plates, which position at the left and right or at the upper and bottom of a member. The stiffeners are steel plates which stiffen steel plates before unified with concrete, and work as shear connectors and structural member after composition. The shear reinforcing steel plates reinforce the steel and concrete sandwich composite plates and are arranged in the member direction and the orthogonal direction. The diaphragm is a minimum space for filled concrete enclosed by the steel plates and the shear reinforcing steel plates.

(2) Characteristics

In recent years, steel plate concrete composite plates combining steel plates and concrete have been widely used as bridge slabs, submerged tunnels, caissons, harbor structures such as floating jetty, rock sheds and so on. The steel and concrete sandwich composite plates is a composite structure that two steel plates and concrete filled between two plates behave as a one piece. Fig.5.3.3 shows the structure after composition and in service and Fig.5.3.4 shows the structure before concrete is filled. This type of composite plates has been applied to submerged tunnels.

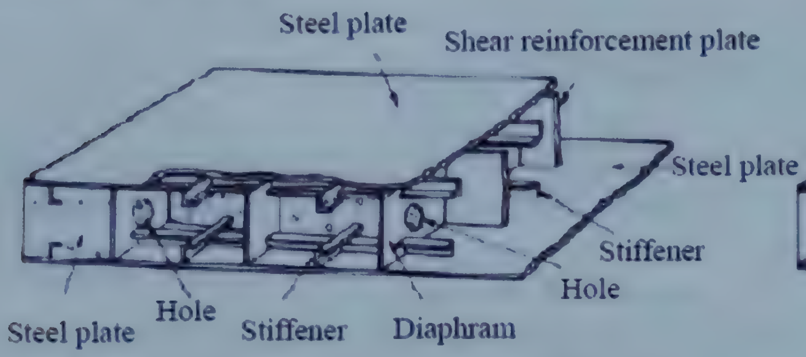


Fig.5.3.3 Example of sandwich composite plate

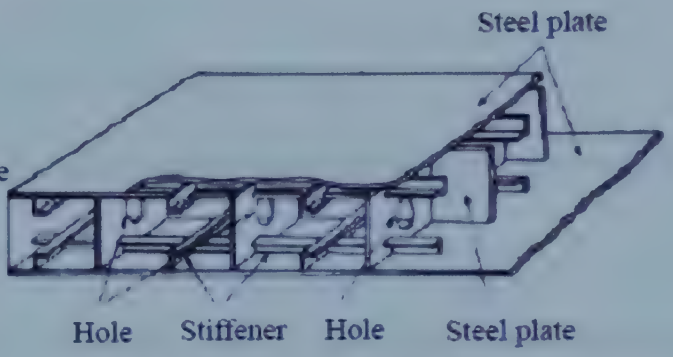


Fig.5.3.4 Example of steel shell structure

(3) Structural analysis

In design of the steel and concrete sandwich composite plates, the same actions as plates of steel structures or concrete structures are generally considered. In this structure, both the construction action with the steel structure before concrete is filled and the in service action after concrete is filled must be considered. Therefore, the construction action and the in service action must be appropriately resisted by structural elements such as bottom plate, rolled steel section, reinforcement and concrete, considering construction stages.

As for structural analysis to obtain responses of the steel and concrete sandwich composite plates, thin- plate theory or beam theory is used depending on actions or characteristics of structures. The effect of nonlinear structural analysis is in principle considered, however, it can be treated as linear if its effect to design response is negligible.

(4) Strength

In the steel and concrete sandwich composite plates, types and amount of actions and resistance mechanism are different at the construction stage with only the steel structure before concrete is filled and the in service stage with composite structure after concrete is filled. Therefore, the strength at both stages must be obtained. As failure types of the steel and concrete sandwich composite plates, there are bending failure, shear failure, local push-out shear failure, and so on. Shear failure after composition is described below.

Although the shapes and arrangement of tensile reinforcing steel plates and shear reinforcing steel plates are different form reinforced concrete members, shear failure of the steel and concrete sandwich composite plates due to out-of-plane shear forces is almost the same as the usual reinforced concrete members. The design shear strength of the steel and concrete sandwich composite plates is classified into following four structural characteristics.

① Design shear strength without shear reinforcing steel plates
② Design shear strength with shear reinforcing steel plates arranged only in the member direction (Fig.5.3.5)
③ Design shear strength with shear reinforcing steel plates arranged only in the vertical member direction (Fig.5.3.6)
④ Design shear strength mixed with shear reinforcing steel plates arranged in the member direction nd those in the vertical member direction (Fig.5.3.7)

In the steel and concrete sandwich composite plates, the shear reinforcing steel plates are arranged in the member direction and also in the vertical member direction, respectively. The shear resistant mechanism of these two cases are much different. As there is no research on shear strength mixed with shear reinforcing steel plates arranged in the member direction and those in the vertical member direction, the method is adopted to determine the shear strength in the safety side.

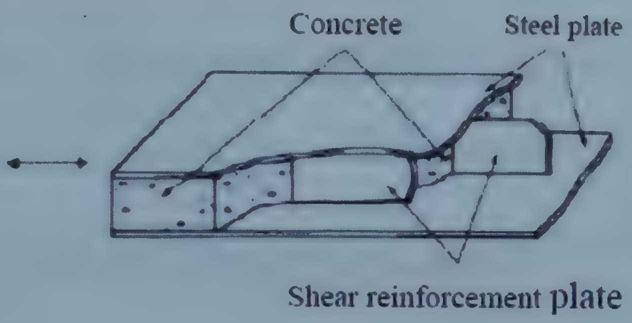


Fig.5.3.5 Shear reinforcement steel plate set in the member direction

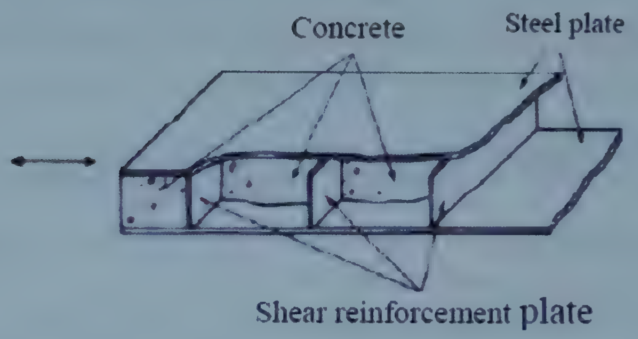


Fig.5.3.6 Shear reinforcement steel plate set in the transverse direction

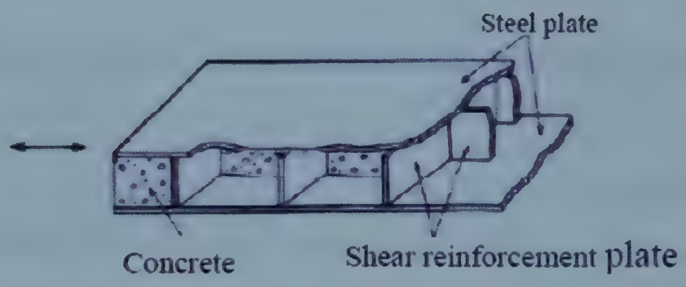


Fig.5.3.7 Shear reinforcement steel plate set both in the member and transverse direction

[Source: Basic Theory and Design of Hybrid Structures -2019-, JSCE]

xii) Design Methodology of Corrugated steel web bridge is extracted from Design Standard of NEXCO Japan, Part II, Bridge Construction

Extracted from Design Standard of NEXCO Japan, Part II, Bridge Construction, NEXCO

Chapter 4 Corrugated Steel Web Bridge

4-1 Scope

This regulation is applied to the design of corrugated steel web bridge.

4-2 Typical Cross Section

Defining the cross section of corrugated steel web bridge shall be execute considering total slab width, cross section of corrugated web, distance of centreline (span of slab), and girder height. It also shall be optimized with span length and total width of bridge.

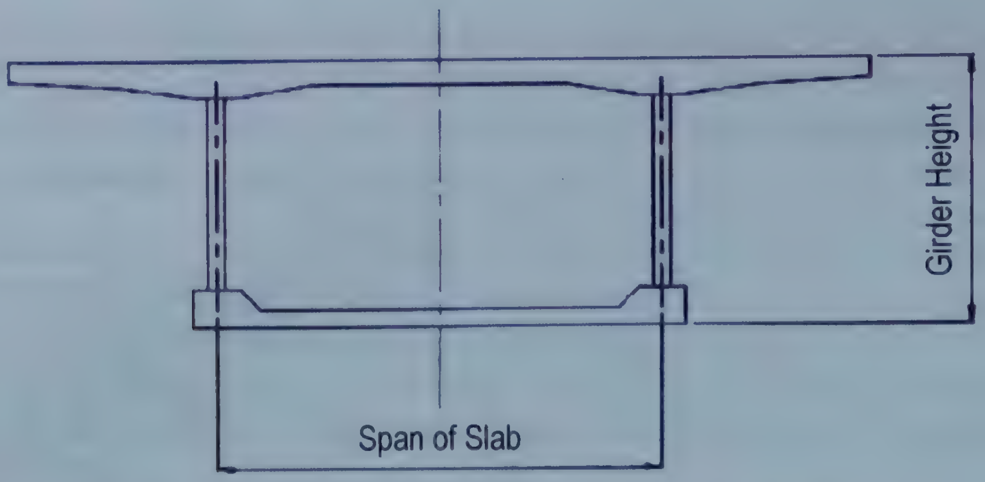


Figure 9-4-1 Typical Cross Section of Corrugate

4-2 Planning

(1) Fabrication and workability of corrugated steel web and transformation method of corrugated steel web shall be considered in designing the corrugated steel web bridge.
(2) Pier cap section (inside of the box-girder cross section) shall be filled with RCC.
(3) Cross beam at the end of the spans and structure and interval distance of diaphragm shall be designed carefully.
(4) Length of the block for cantilever erection shall be set considering the wave length of corrugated steel web.

4-4 Basic Policy of Design

(1) Design loads and combination of the loads shall be the same as the typical PSC box-girder bridge
(2) Design shall be done by design load and ultimate load.
(3) Safety of the bridge by seismic load and the combination with other load and seismic load shall be checked.
(4) Corrugated steel web shall have enough strength for preventing sheer buckling in the event of maximum design earthquake.

4-5 Structural Analysis

Developing model(s) for structural analysis shall be done following the applied methodology of analysis for each member.

(1) Members are modelled as beam member in frame analysis.
(2) Appropriate rigid section shall be considered at the joint section of rigid structure and section where the cross section changes considerably in frame analysis.
(3) Panels which bear only shear flow, and members which is located at the edges of the panels that are beam members are combined in continuous shear flow panel analysis.
(4) Shell elements and solid elements are combined in 3-dimentional FEM analysis based on the members and their structure.

4-6 Transversal Analysis

(1) Transversal analysis shall be done by 2-dimensional frame analysis, 3-dimensional FEM analysis or other equivalent method.
(2) Sectional force by dead load or prestress could be calculated by analysis model which assumes the box-girder cross section as rigid structure formed by web and top/bottom slab.
(3) If the span of slab is 6m and shorter, sectional force by live load could be calculated with the length of the span of slab as the distance of centroid axes of corrugated steel web except for the bending moment of span. Effect by small transversal rigidity of corrugated steel shall be appropriately considered in calculating bending moment of span.

4-7 Design of Main Girder

(1) Following items shall be checked in design of main girder.
  1) Safety in loading design loads and combination of loads
  2) Safety in loading ultimate loads and combination of loads
  3) Safety against fatigue
  4) Safety in construction
(2) Wave length and amplitude of corrugated steel web shall be set appropriately considering its buckling strength, layout and transportation condition.
(3) Thickness and welding size of corrugated steel shall be fabricated size which satisfies corresponding standards.
(4) Following structural details shall be carefully designed in design of corrugated steel web.
  1) Radius of bending corrugated steel web in fabrication
  2) Jointing corrugated steel web
  3) Reinforcement by cement concrete
  4) Attachment of additional members
  5) Additional manholes

4-8 Design of Pier Cap

(1) Cement concrete shall be casted inside or corrugated steel web around pier cap in order to reinforce the capacity against buckling and relieve local stress.
  1) Connection of corrugated steel web and cement concrete is by stud dowels.
  2) Minimum thickness of cement concrete inside corrugated steel web is 200mm.
(2) Cement concrete shall be design with the assumption that the shear is distributed to corrugated steel web and cement concrete in proportion of rigidness against shear of both materials. Corrugated steel web shall be designed with the assumption that corrugated steel web bear entire shear.
(3) When considering the crack caused by temperature stress, binding effect by corrugated steel web shall be considered.
(4) Additional bending caused by binding effect of cross beam and/or slab with design loads and their combinations shall be considered.
(5) Rebar volume in the cross section shall be calculated against the shear born by cement concrete inside corrugated steel web with ultimate loads and their combinations. Corrugated steel web shall be checked against distributed shear.

4-9 Design of Slab

(1) Slab shall be designed carefully considering the mechanical property of cross section of box-girder with corrugated steel web.
(2) Total thickness of PSC slab shall be the same as the PSC slab without corrugated steel web.
(3) Crack shall not be permitted with dead load only and within limit state of cracking with live load loaded. Rebar against tension caused by live load shall be arranged in the slab.
(4) Slab shall be checked in terms of bending moment and axial load with ultimate load and their combinations.

4-10 Design of Cross Beam and Diaphragm

(1) Cross beam and diaphragm shall have sufficient rigidity and be arranged with appropriate interval.
(2) Designing with design loads and their combination shall be done with the same methodology of PSC box-girder.
(3) Designing with ultimate loads and their combination shall be check in bending moment, axial force, and shear.
(4) Reinforcement by diaphragm shall be considered when the bottom slab has bend angle.

4-11 Connecting Section of Corrugated Steel Web and Slab

4-11-1 Structure of Connecting Section of Corrugated Steel Web and Slab

(1) Connecting section of corrugated steel web and slab shall have structure which surely transfer the horizontal shear in longitudinal direction. It also shall bear the bending moment of slab in transversal direction caused by live load and surely forms box cross section.

(2) Appropriate corrosion prevention treatment shall be done to the connecting section in order to perform its function throughout the service life. It also shall have the durability against fatigue caused by live load.

(3) Connecting section shall bear all the loads and combinations. Structural property, workability, durability, and etc. shall be considered in selecting the structure of connecting section.

4-11-2 Design of Connecting Section of Corrugated Steel Web and Slab

(1) Safety in loading design load and ultimate load and their combinations shall be checked against longitudinal horizontal shear at the connecting section of corrugated steel web and slab.

(2) Safety in loading design load and ultimate load and their combinations shall be checked against transversal moments by loads at the connecting section of corrugated steel web and slab.

4-11-3 Design of Welding Section

Design of welding section shall be done against the combined stress of longitudinal and transversal directions by design loads and their combinations.

4-11-4 Structural Details

(1) Details shall be designed considering the load bearing function of each connecting section.

(2) Thickness of the members of connecting section shall be designed considering the performance of the exiting bridge and welding property.

4-12 Connecting Section of Corrugated Steel Web and Cross Beam

4-12-1 Structure of Connecting Section of Corrugated Steel Web and Cross Beam

(1) Connecting section of corrugated steel web and cross beam (pier cap, cross beam at the girder ends) shall have structure which surely transfer vertical shear caused by longitudinal bending shear movement

(2) Appropriate corrosion prevention treatment shall be done to the connecting section in order to perform its function throughout the service life. It also shall have the durability against fatigue caused by live load.

(3) Connecting section shall prove its safety by laboratory experiment. Structural property and economy shall be considered in selecting the structure of connecting section.

4-12-2 Design of Connecting Section of Corrugated Steel Web and Cross Beam

(1) Safety in loading design load and ultimate load and their combinations shall be checked against vertical shear caused by longitudinal bending shear movement at the connecting section of corrugated steel web and concrete cross beam.

(2) Safety shall be checked by shear force loaded to connecting section is below allowable shear or ultimate bearing capacity of slipping stopper.

4-12-3 Design of Welding Section

Design of welding section shall be done against the vertical shear stress by design loads and their combinations.

4-12-4 Structural Details

Size of the members of connecting section shall be designed considering the bearing property against loads.

4-13 Anchorage and Deviator of Outer Cable

(1) Anchorage of outer cable shall have the appropriate structure which have the function to transfer tension of the cable to main girder smoothly considering the structure of main girder, property and number of the outer cable. Design of anchorage shall secure bearing capacity of the load and durability evaluating the stress caused by the tension force of outer cable.
(2) Deviator of the outer cable shall have the structure which secure the function of deviator which sustain the main girder. Design of deviator shall secure bearing capacity of the load and durability evaluating the stress caused by the pressure or outer cable.
(3) Load and stress of the anchorage and deviator are very complicated. Careful study and evaluation is necessary in designing.

[Source: Design Standard of NEXCO Japan, Part II, Bridge Construction, NEXCO]

3) Mixed Superstructure

i) Types of mixed superstructure

There are approximately four types of mixed structures defined by JSCE such as,

- Hybrid rigid frame bridge
- Hybrid cable-stayed bridge
- Mixed continuous girder bridge
- Hybrid arch bridge

Hybrid rigid frame bridge

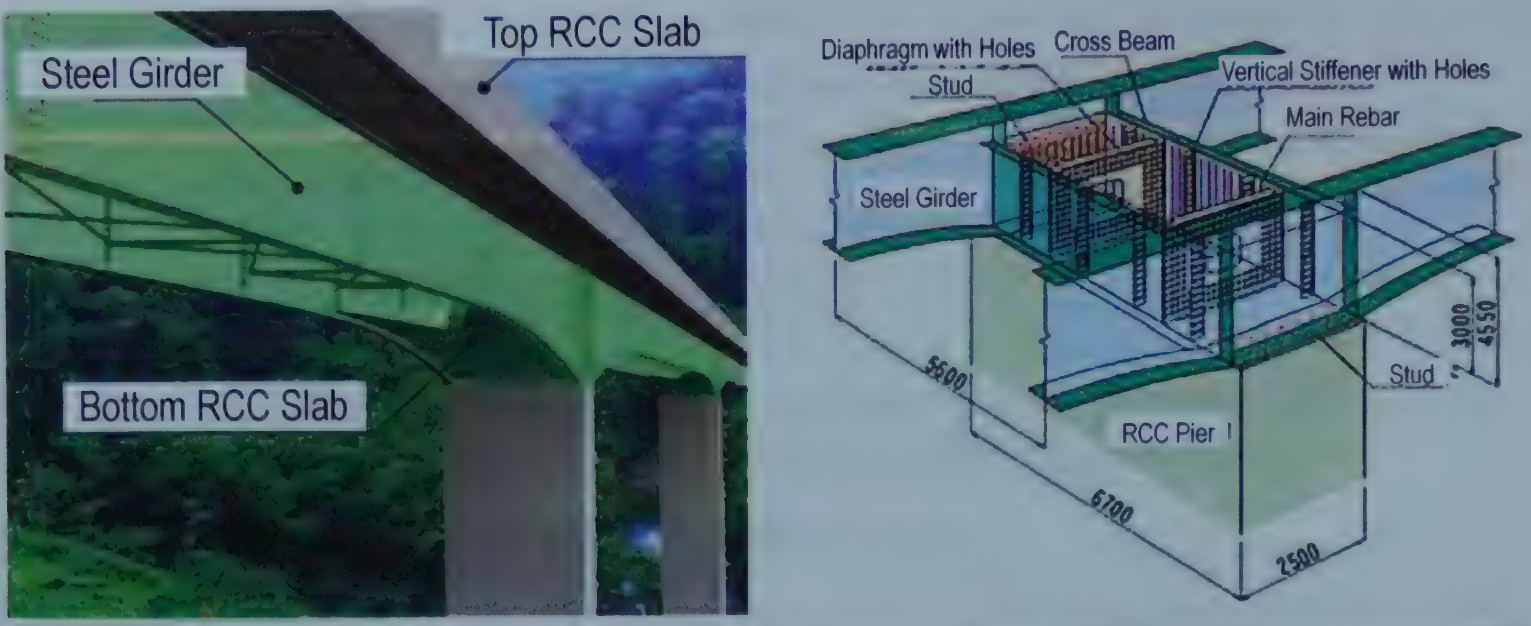
Mixed continuous girder bridge

ii) Suitable span lengthen by composite superstructure is shown on table below

Table 3-11: Suitable Span Length by Composite Superstructure

Type of superstructure	Span length
Hybrid rigid frame bridge	80 to 110 m
Hybrid cable-stayed bridge	120 to 500 m
Mixed continuous girder bridge	130 to 180 m
Mixed continuous girder bridge (extra-dosed bridge)	150 to 275 m
Hybrid arch bridge	200 to 320 m

iii) Hybrid rigid frame bridge has steel plate girder superstructure rigidly connected to RCC substructure. It usually has 2 main plate girders and it has diaphragms and vertical stiffeners connected to cross beam at the top end of RCC substructure. Steel materials and RCC is connected by studs which transfer force between superstructure and substructure.



(Source: https://www.jasbc.or.jp/technique/document/)

Figure 3-5: Typical Structure of the Top End of RCC Pier with Steel Plate Girder Rigidly Connected

iv) For reference Chapter 6, Basic Theory and Design of Hybrid Structures -2019-, JSCE is quoted as follows,

Quoted from Basic Theory and Design of Hybrid Structures -2019-, JSCE

Chapter 6 Design of Mixed Structure

6.1 Introduction

A mixed structure is a structure where more than two different types of members such as steel members, concrete members and various composite members are connected. In mixed structures, constituting members can be chosen depending on usage conditions and objectives, and consequently the strength, stiffness and ductility of different types of members are effectively utilized. Therefore, mixed structures have attracted attentions and actively been applied. Mixed structures also give design engineers large freedom of structural planning and design. The performance of mixed structures depends on that of connections which should be designed as an independent member in a structure.

Suppose that the connection of the RC member and the steel member is subjected to bending moment and shear force as shown in Fig.6.1.1. When the connection is subjected to shear force, as it resists the force to slip in the vertical direction, bond action is required at the connection. This bond action can be thought to be the same as that between concrete slab and steel girder in the composite beam. In Fig.6.1.1, the headed studs attached on the end of the steel member resist this shear force. When the connection part is subjected to a positive bending moment, bearing forces act on the upper side of members where the concrete member and the steel member push each other, whereas on the tension side they move away so as to produce a gap between them. Therefore, on the compressive side of a boundary plane, countermeasures such as use of thicker steel plates to widen the bearing area is necessary so that the end of concrete member will not crush nor stress concentration does not occur at the steel plate. On the other hand, on the tensile side of the boundary plane, bond action should be added, for example by introducing pre-stress forces with PC bars, to resist tensile forces.

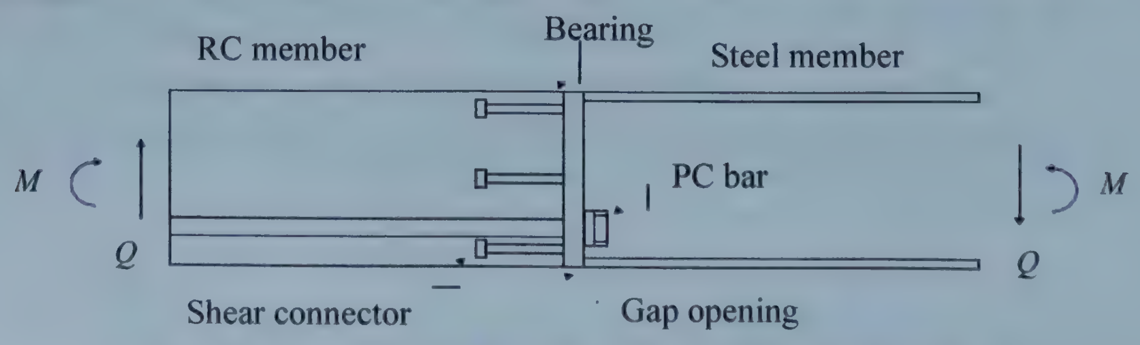


Fig.6.1.1 Connection of steel member and RC member

6.2 Classification of connections

Connections indicate the whole area which is necessary to transfer sectional forces between two members. The connections of members of different types can be between members such as girders and columns, columns and piles, corner part of box-type structures, girder and girder, pier and footing, and so on (Fig. 6.2.1). Some of the connected structures are introduced below.

(1) Classification by materials of connected members

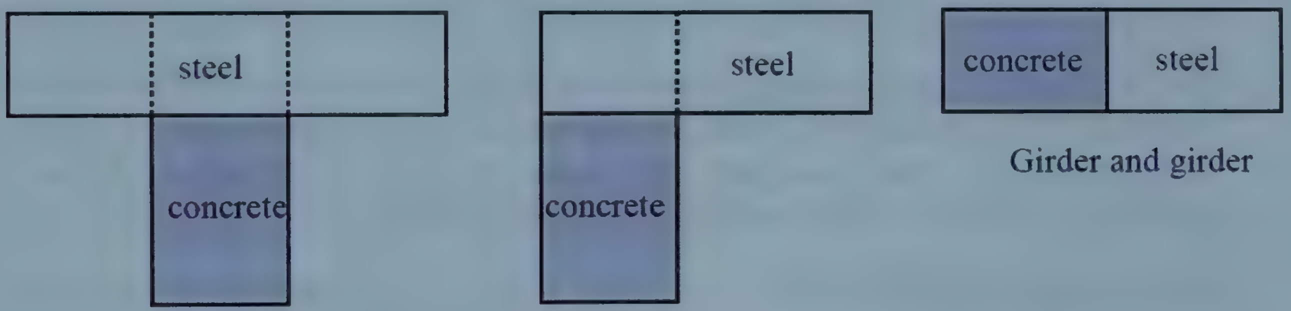
- Connection between steel members and RC members
- Connection between SRC members and RC members
- Connection between steel pipe with concrete members and RC members
- Connection between PC members and RC members

(2) Classification by direction of connection

- Series type: Two different types of members are connected in series.
- Perpendicular type: Connection of two different types of members whose member axis are crossed at right angles, such as horizontal member and vertical member, or lateral member and vertical member.

(3) Classification by force transfer property of connection (rotational rigidity of connection)

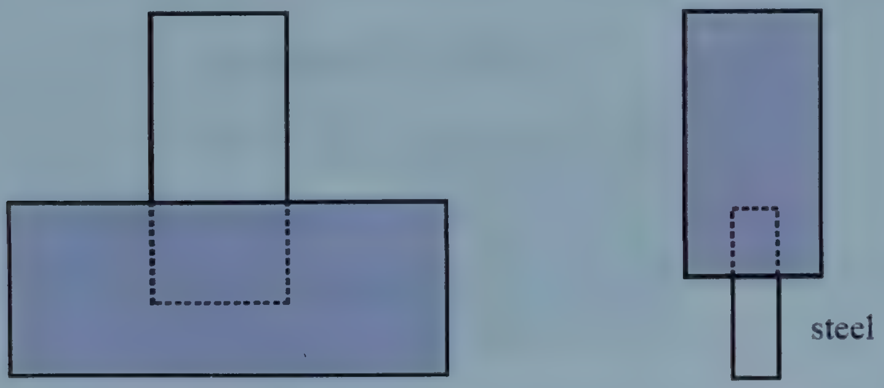
- Rigid connection: Bending moment and shear forces are transferred. The effect of rotation of the member on the distribution of sectional forces is negligible.
- Pin connection: Only shear forces are transferred. It possesses sufficient rotational capacity.
- Intermediate connection: Bending moment and shear forces are transferred. The effect of rotation of the member on the distribution of sectional forces is not negligible.



Girder and pier

Corner of rigid frame

Girder and girder



Pier and footing

Pier/Abutment and girder

Fig.6.2.1 Application of connections of different types of members

6.2.1 Beam to beam

Suppose that the RC beam is connected with the steel beam in the longitudinal direction, as shown in Fig. 6.2.2. When the members are subjected to axial forces, shear forces and bending moments, these forces must be smoothly transferred at the connection. The basic design method in the case shown in Fig. 6.2.2 is described in this section.

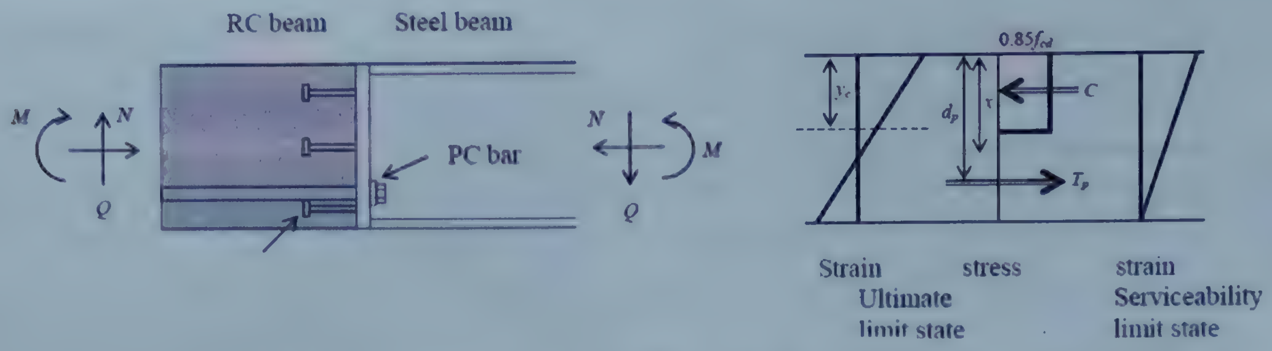


Fig.6.2.2 Connection of steel beam with concrete beam

In this kind of structure, the axial strength is not usually critical. Therefore, let us first evaluate the bending strength under the compressive axial forces including pre-stress forces. On the compression side, the end of the steel member and concrete are pressed each other in the connected plane, by which stresses are transferred. Therefore, using the same evaluation method for RC member and assuming the strain and stress distribution due to compressive axial force and bending moment as in Fig. 6.2.2, the bending strength Mu is obtained as follows.

$$M_u = T_p\left(d_p - y_c\right) + C\left(y_c - \frac{0.8}{2}x\right) \qquad (6.2.1)$$

where, T_p: additional axial forces in pre-stressing bars caused by bending moments, C: total forces due to compressive stresses in concrete, and yc: a position of centroid of cross-section. The position of the neutral axis x is obtained by the following equilibrium relation.

$$N = C - T_p \qquad (6.2.2)$$

As for the serviceability condition, assuming that no gap occurs on the tensile side, the limit value of bending moment is obtained based on the stress distribution where all the section is in compression (Fig.6.2.2).

On the other hand, when bending moment acts at the connection between the steel beam and RC beam, the connection may be designed, where bending moment can be resisted by a couple of forces, as shown in Fig. 6.2.3. In this case, shear connectors must be attached to transfer forces, and the bending strength Mu can be obtained by the shear force T working at the shear connector part.

$$M_u = T \cdot h = n \cdot Q_a \cdot h \qquad (6.2.3)$$

where, Qa: shear strength of a shear connector, and n: number of shear connectors.

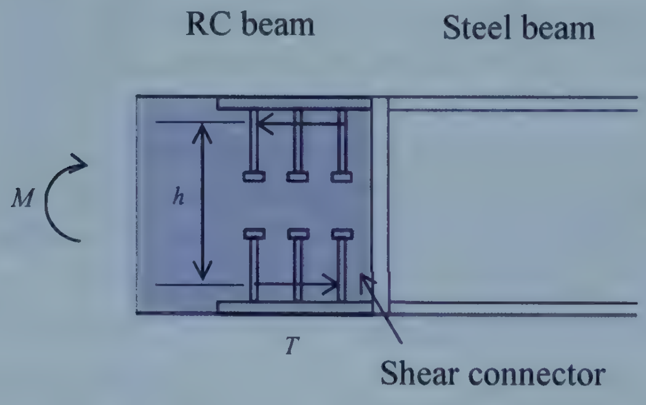


Fig.6.2.3 Transfer of bending moment at connection of steel beam with concrete beam

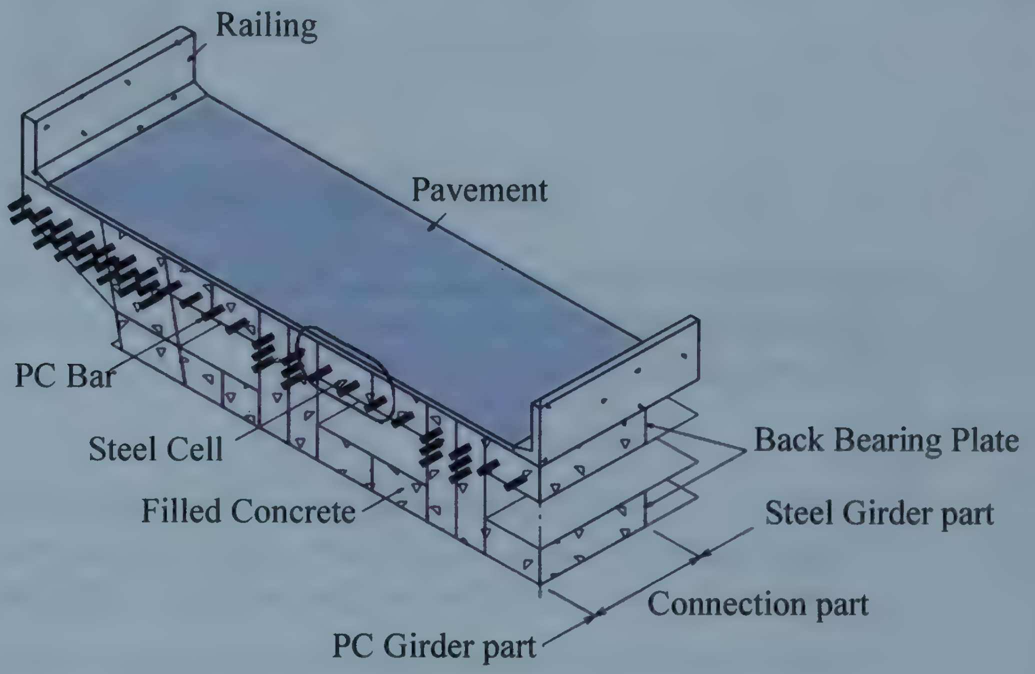
When shear forces act at the connection of the steel beam and the RC beam, shear connectors are attached on the contact plane of both members to resist the external shear forces. The shear strength Qu in this case is obtained as below.

$$Q_u = n \cdot Q_a \qquad (6.2.4)$$

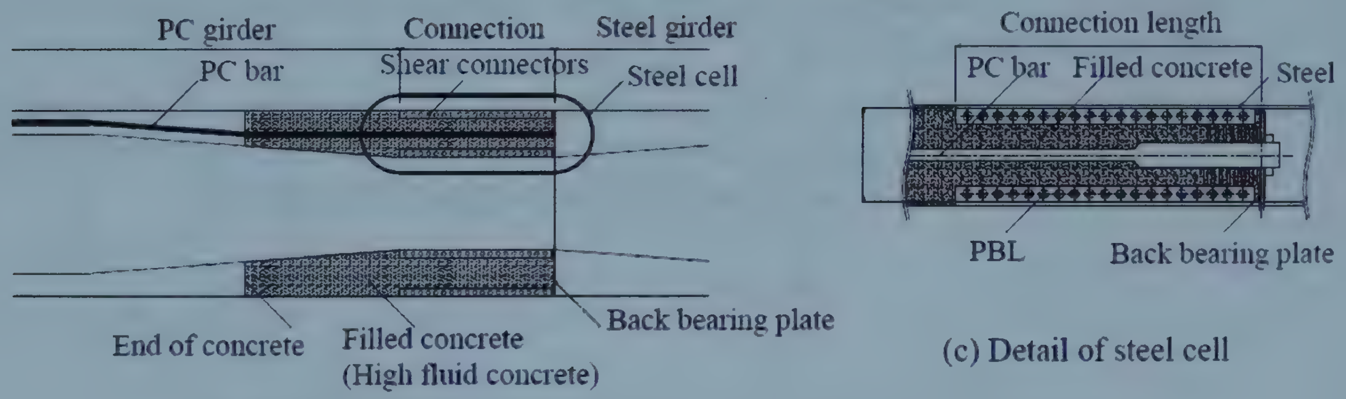
Next, the connection of the steel girder and the pre-stressed concrete girder (referred to as PC girder hereafter) is explained with Fig. 6.2.4, where these girders are connected in series in the axial direction by using concrete-filled steel cells with shear connectors and PC bars. The steel girder has multi-cells at the connection which work as the anchor structure. The PC bars are anchored at the back plate of the steel cells. Shear connectors are attached to the steel cells which are filled with concrete.

Another type of connection is shown in Fig. 6.2.5, where the steel girder and the PC girder are connected with the bearing connection. In this connection type, shear forces are transferred by the friction forces caused by bearing forces at the connection. Therefore, no shear connector is used at the connection plane or inside steel cells. The connection between the steel girder and the PC girder consists of the in-situ concrete part and the bearing part of steel structure filled with concrete. The connection is unified by introducing pre-stress forces by PC bars, and shear forces are transferred by friction forces between the front plate of the bearing part and the in-situ concrete.

There are three types of failure of shear keys: ① bending failure, due to main steel in tension yields at the connection section, buckling of main steel in compression, or compressive failure of concrete, ② shear slip at the connection section or shear failure of shear keys, and ③ bearing failure of concrete contacting steel plates.



(a) Overview of Connection



(b) Sideview of Connection

(c) Detail of steel cell

Fig.6.2.4 Example of connection of steel girder with PC member

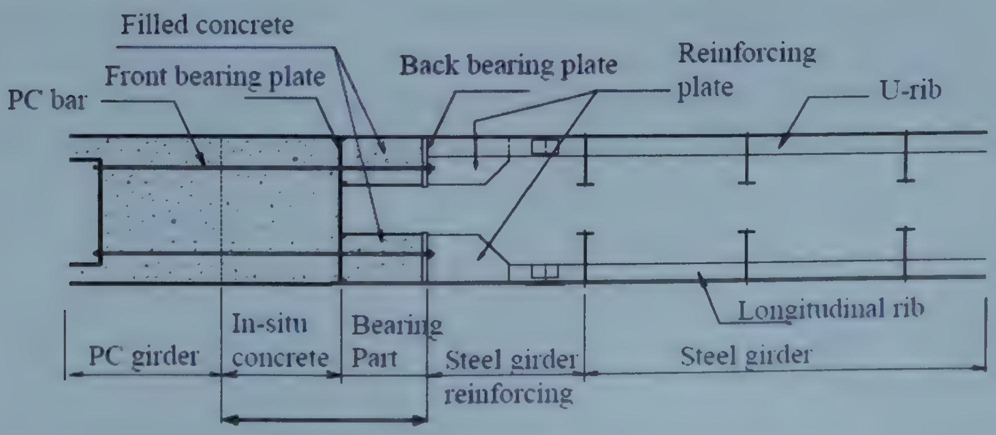


Fig.6.2.5 Example of bearing connection of steel girder with PC member

6.2.2 Column to beam

When the steel girder and the RC column are rigidly connected and subjected to shear force and bending moment, as shown in Fig. 6.2.6, the rigid connection must resist these sectional forces.

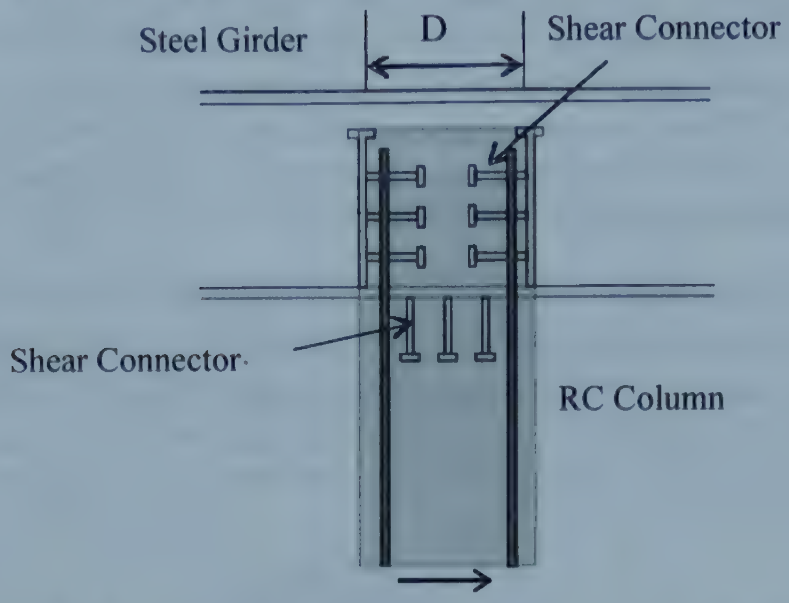


Fig.6.2.6 Example of connection of steel girder with RC column

The bending moment between the RC column and the steel girder is resisted by the shear connectors attached on the web of the cross beams. The bending strength of the rigid connection is expressed by the following equation.

$$M_u \square n \square Q_a \square D \qquad (6.2.5)$$

where, D : distance between cross beams, Q_a : shear strength of a shear connector, and n : number of shear connectors. In a structure shown in Fig. 6.2.6, the force transferred by shear connectors on the web is transferred to steel bars in concrete inserted in the steel girder, then finally transferred to the RC pier.

On the other hand, the shear force is transferred from the steel girder to the RC pier by shear connectors attached on the lower flanges of the steel girder. Therefore, the shear strength Qu can be calculated by the same equation as Eq. (6.2.4).

Fig.6.2.7 shows an example of this type of connection. The steel girder is connected to the vertical RC pier. The top part of the RC column is enclosed by the steel girder and the cross beams. Diaphragms are also set between cross beams to increase the confinement degree of filled concrete.

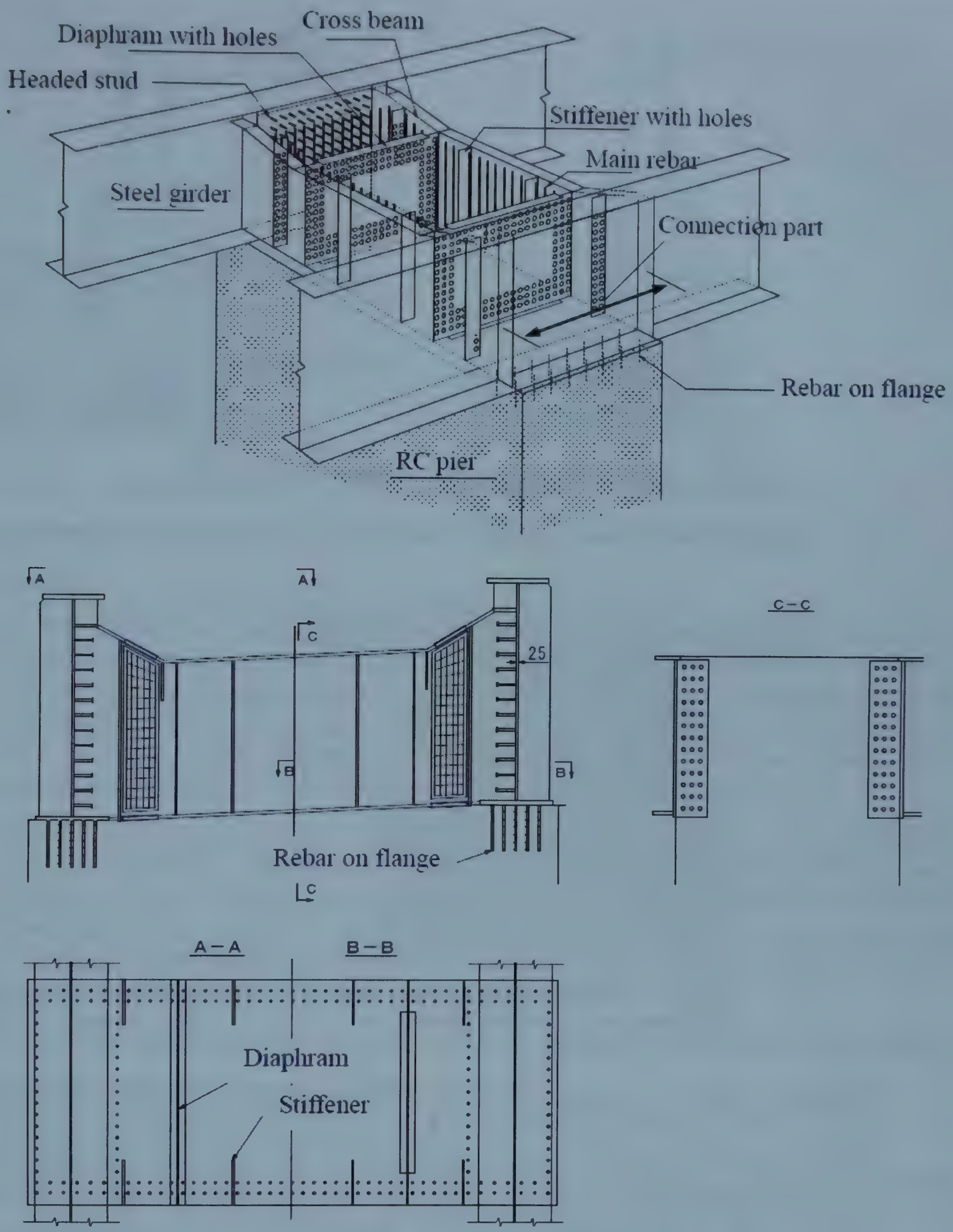


Fig.6.2.7 Example of connection of steel girder with RC pier

6.2.3 Column to foundation

When steel columns are fixed to concrete footing, as shown in Fig.6.2.8 and Fig.6.2.9, the connection is subjected to axial force, shear force and bending moment. The type of Fig.6.2.8 is called "RC type", and that of Fig.6.2.9 "pile type". The compressive and tensile forces are resisted by concrete under the base plate and anchor bolts, respectively, in the RC type. Both compressive and tensile forces are resisted by anchor bolts in the pile type[1).]

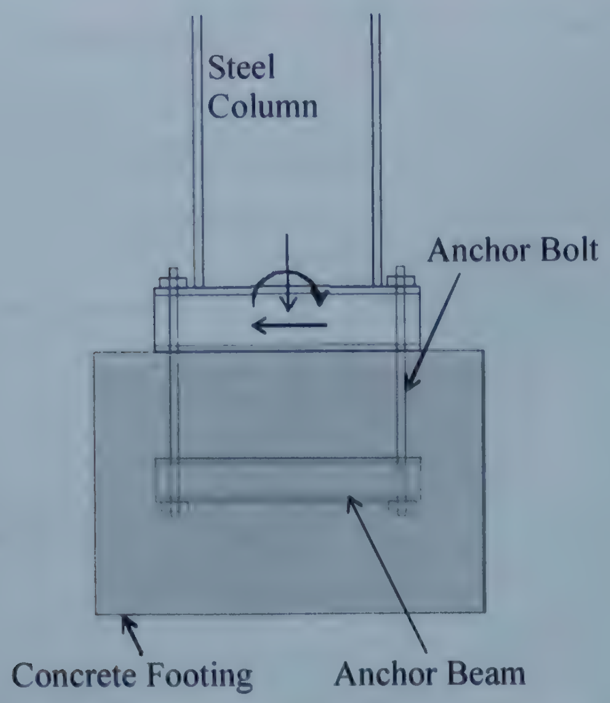


Fig.6.2.8 Anchor frame type (RC type)

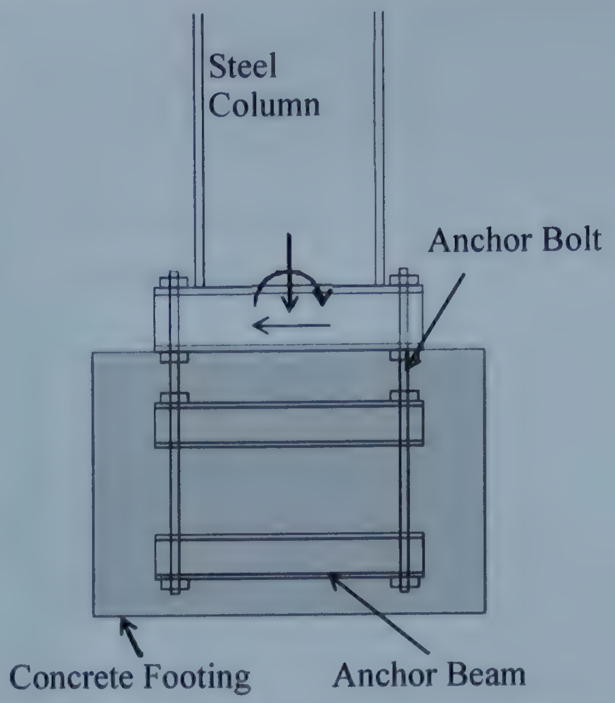


Fig.6.2.9 Anchor frame type (Pile type)

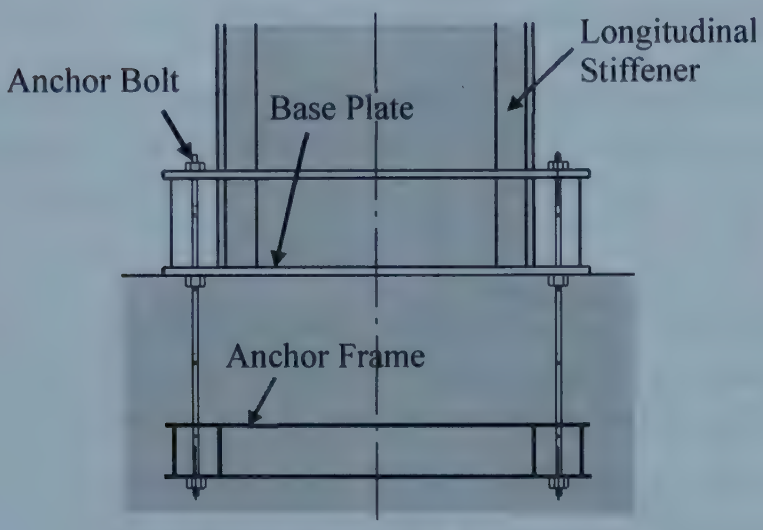


Fig.6.2.10 Example of connection of CFT member by anchor frame type

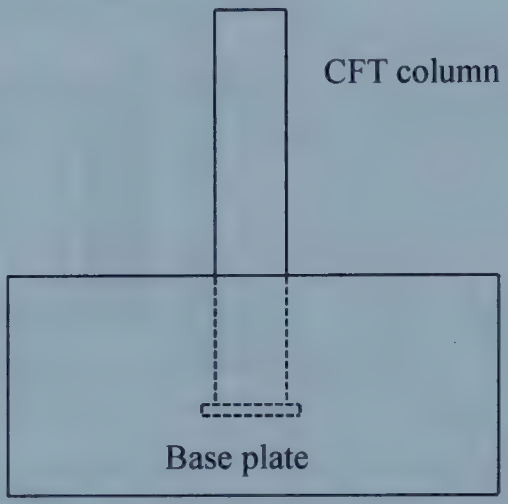


Fig.6.2.11 Example of connection of CFT member by embedded type

Strengths of section, necessary for designing these two types of connections, are explained here. When the anchor section of the RC type of Fig. 6.2.8 is subjected to axial force *N* and bending moment *M*, the RC method is applied with the concrete in compression and the anchor bolts considered as steel bars. Therefore, the bending strength can be obtained by the same idea as Eq. (6.2.1), where T_p is replaced by the axial force of anchor bolts. For the pile type of Fig. 6.2.9, neglecting the compressive resistance of concrete, only anchor bolts resist axial forces and bending moments. Therefore, the bending strength of a rectangular section is obtained as follows:

$$M_u = n \cdot N_u \cdot D \qquad (6.2.6)$$

where, N_u: yield force of an anchor bolt on the compression side, n : number of anchor bolts on the compression side, and D : a distance between centroids of anchor bolts in compression and tension.

Shear forces are resisted by only anchor bolts in both types. Shear strength of anchor part Q_u can be obtained by Eq.(6.2.7), where Q_a: shear strength of an anchor bolt, and n : number of anchor bolts.

$$Q_u = n \cdot Q_a \qquad (6.2.7)$$

Fig. 6.2.10 shows the anchor frame type used for the connection of concrete-filled steel columns to footing, Fig. 6.2.11 is the embedded type with the concrete-filled steel pipe column directly embedded into the RC footing, and Fig. 6.2.12 is the socket type with the concrete-filled steel column inserted into the steel pipe pile or the sockets of steel pipe filled with concrete.

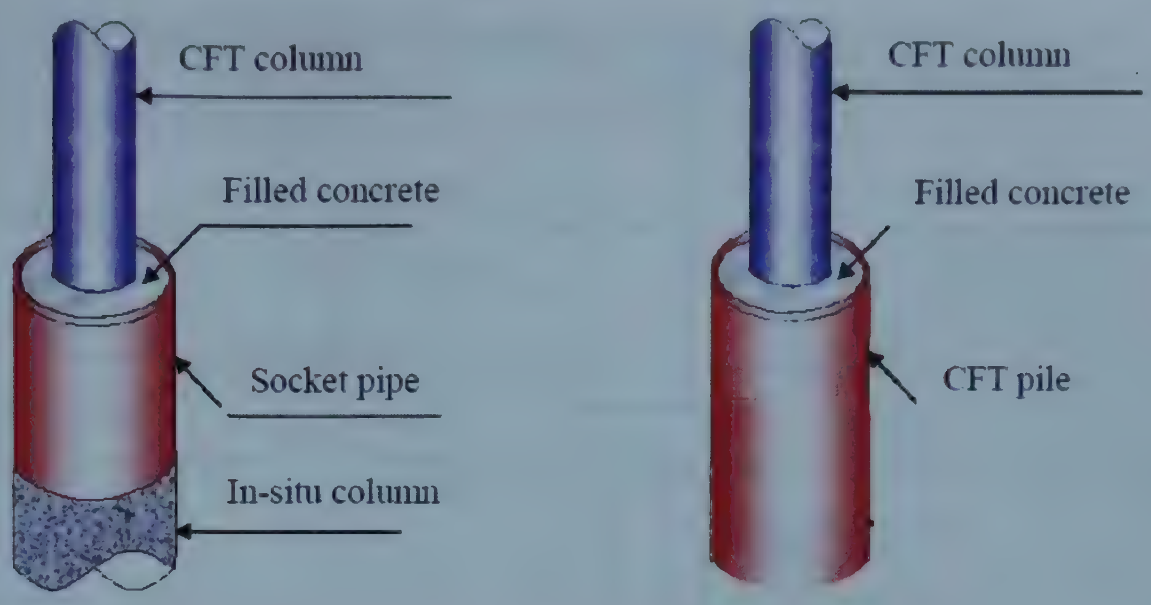


Fig.6.2.12 Connection of steel member with RC member

6.2.4 Jointless structures at bridge abutments

(1) Introduction

As the number of bridges with an age over 50 years is increasing, and a large amount of budget is needed for repair and reinforcement, it is required to reduce the cost of maintenance. In addition, considering the life cycle cost, not only the maintenance cost but also the initial cost should be reduced. At present, many bearings and expansion joints are adopted in Japanese bridges. As bridges are getting older, damages and degradation are found at those bearings and expansion joints, creating a serious problem. This leads to jointless structures at bridge abutments which requires no bearing nor expansion joint.

A jointless structure at bridge abutments is a structure that the girder and the abutment are rigidly connected at the girder ends eliminating bearings and expansion joints, which can reduce the initial cost and also the maintenance cost. On the other hand, deformation of the girder is absorbed by bearings and expansion joints in the conventional girder bridges. Therefore, there are some problems to be solved, such as alternative functions to absorb girder deformation, clarification of force transfer mechanism at the rigid connection, and long-term behaviour of the jointless structure at a bridge abutment.

In this section, classification and characteristics of jointless structures at bridge abutments are described.

(2) Classification of jointless structures at bridge abutments

a) Rigid frame structure

In this structure, deformation of the girder is restrained by the rigid substructures or foundations. The superstructure and substructure are rigidly connected at the abutment, as shown in Fig. 6.2.13, where bearings and expansion joints are eliminated, but the foundation has the same structure as the conventional abutment. This type is applied to bridges with a span length of 50 m or less and an abutment height of 15 m or less2).

b) Integral abutment structure

In this structure, deformation of the girder is absorbed by flexibility of the abutment foundation. The superstructure and substructure are rigidly connected at the abutment, as shown in Fig.6.2.14, but the piles are arranged in one row so that the abutment can deform flexibly. This type is applied to bridges with a span length of 40 m or less and an abutment height of 10 m or less2), 3).

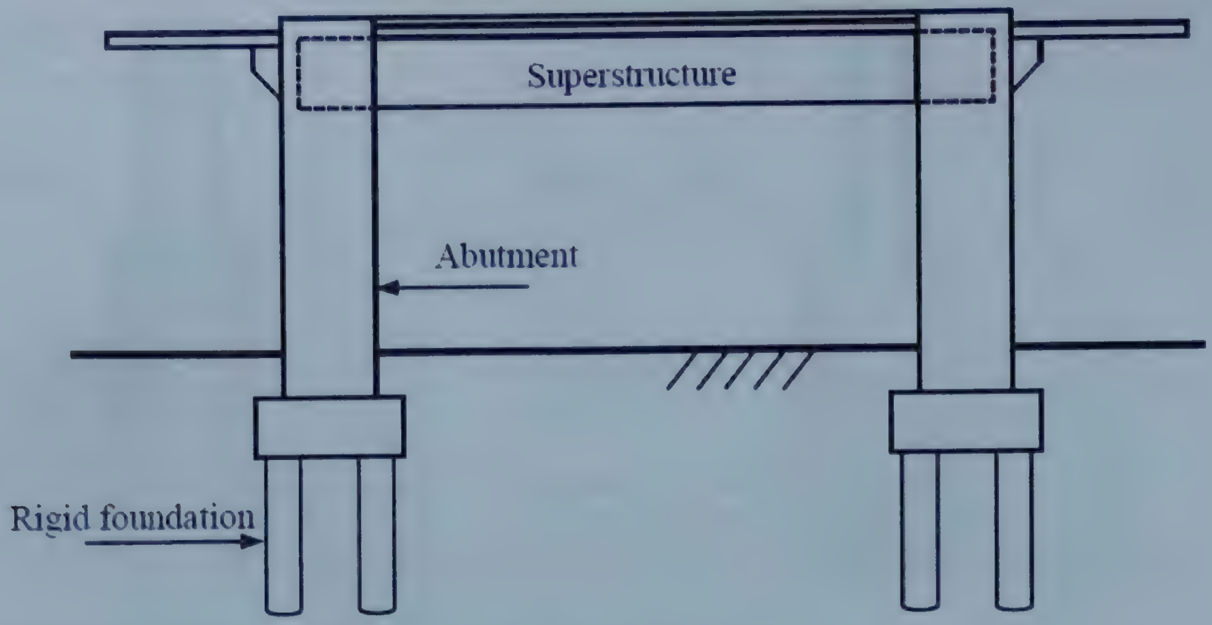


Fig.6.2.13 Rigid Frame Structure

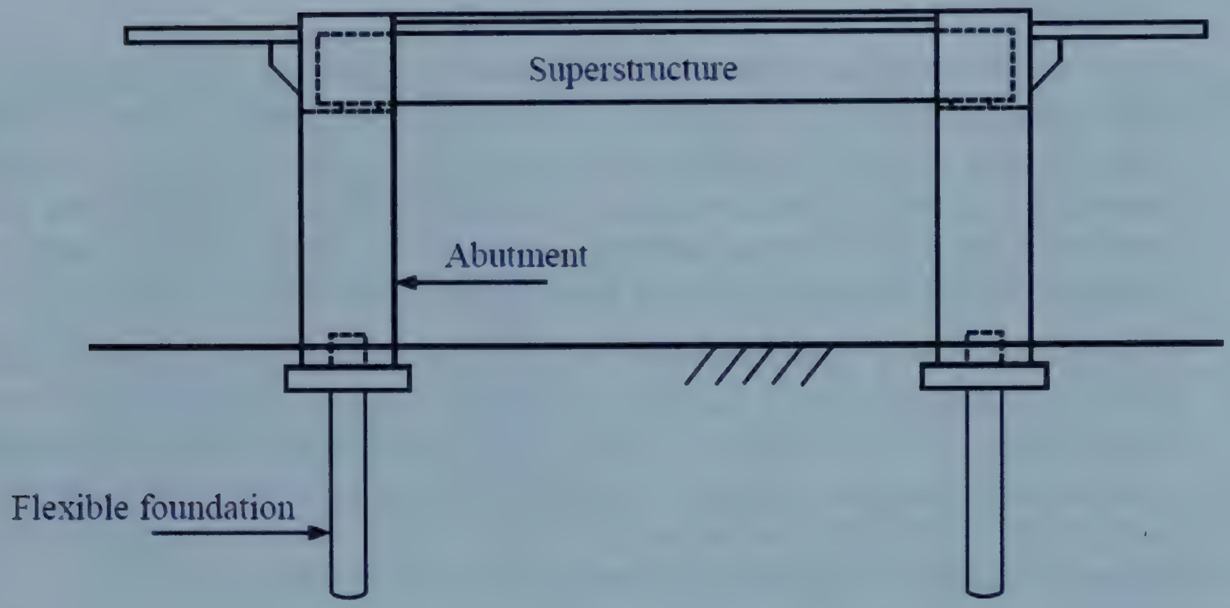


Fig.6.2.14 Integral Abutment Structure

(3) Advantages of jointless structures at bridge abutments

Many of bearings and expansion joints are adopted in Japanese bridges. However, they have some problems at the girder end such as malfunction of a drainage system and corrosion of bearings. Advantages of jointless structures at bridge abutments are as follows.

a) Economy

Bearings, expansion joints and unseating prevention devices are unnecessary, which reduces the initial cost. Maintenance works for the equipment are also unnecessary, which reduces maintenance cost.

b) Drivability

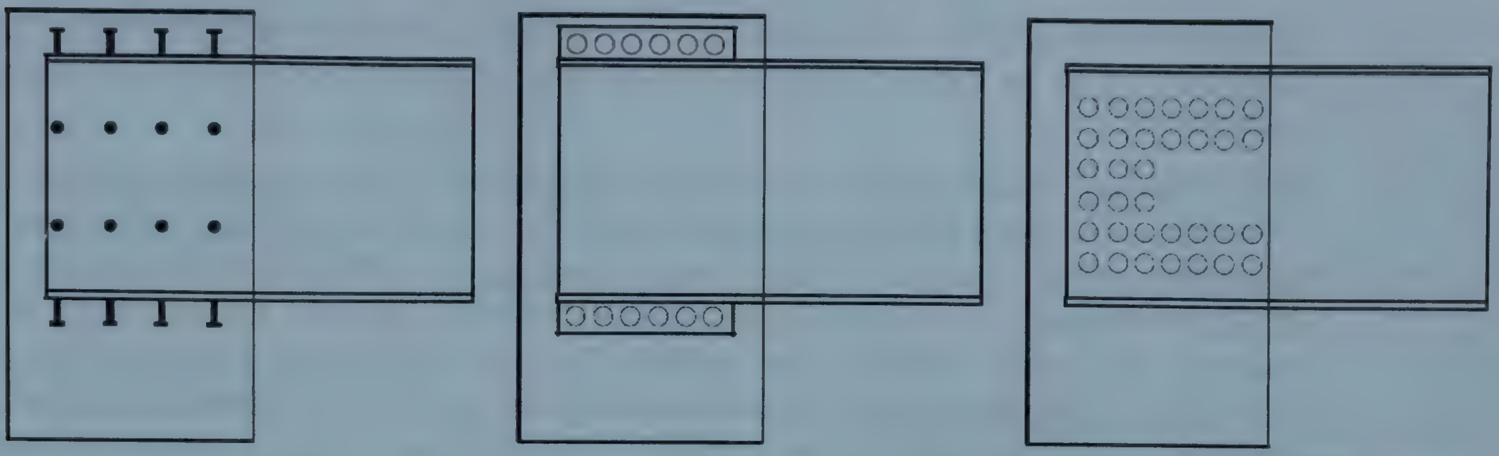
Drivability is improved because superstructure and substructure are rigidly connected.

c) Seismic resistance

Risk of bridge fall at earthquake is reduced by integral pier caps between superstructures and substructures. Deflection is smaller than the conventional bridge structure.

(4) Corner structure

Design specification on jointless structures at bridge abutments was first issued by the Specifications of highway bridges, IV, Sub-structure[1], in 2012. However, structural details are not clearly specified at practical levels such as the length of embedded length or the force transfer mechanism. Therefore, research and experiments on the corner section have been conducted in Japan and various structures have been proposed and constructed. Some of them are introduced in this section (Fig. 6.2.15) 4).



(a) Rigid connection with headed studs

(b) Rigid connection with PBL

(c) Rigid connection with holes in the web

Fig.6.2.15 Structures at corner4)

a) Rigid connection with headed studs

The steel girder is embedded into the concrete of the bridge abutment. Headed studs are welded on the upper and lower flanges and the web of the steel girder at the corner.

b) Rigid connection with PBL

The steel girder is embedded into the concrete of the bridge abutment. PBLs are welded on the upper and lower flanges and the web of the steel girder at the corner.

c) Rigid connection with holes on the web

Holes are made in the web of the steel girder embedded into the concrete, which work as shear connectors. This method eliminates PBLs and improves economy and constructability.

6.3 Design of connections of different types of members

6.3.1 Basis of design

Performance of mixed structures consisting of different types of members is greatly affected by performance of the connection of different types of members. The connection is taken as a structural part and should be independently designed and verified. Mixed structures have connections with more than two different types of members such as steel members, concrete members and various composite members. The common verification method of the connection in a mixed structure is not easily established because different types of members and structures have many freedoms.

As the connection of different types of members affects safety of the whole structure, the connection should have sufficient performance so that the whole structure can show the required performance. Force transfer properties (rotational rigidity) at the connection is classified to the rigid connection or the pin connection, which transfers sectional forces such as axial forces, bending moments and shear forces. To connect two different types of members at a section, a structure to transfer sectional forces to each other is necessary, and it also requires the stress transfer area including anchors and shear connectors. As there may be a sharp stress change in the member or stress concentration for a certain connection type, the stress distribution area may be necessary to decrease its effect. The stress transfer area and the stress distribution area are defined as the connection of different types of members. The equipment to transfer sectional forces and transfer mechanism is called as transfer elements, including mechanical force transfer with shear connectors, force transfer by bond, and force transfer by bearing and friction5).

In a structural system with the connection of different types of members, it is necessary to determine the order for the connection and the general part to reach limit state conditions in order to satisfy the required performance.

As for serviceability, stress in the materials of the connection should be within an elastic range and transfer forces working on the transfer elements such as shear connectors and bearing should also be within an elastic range.

As for safety, the connection should not fail before the other parts in the connected members. This is because, in the connection, there are many unknowns about nonlinear behaviours and deformation performance, a failure mode of the connection is brittle in many cases, and it is difficult to repair and strengthen when it is damaged. Whereas, in the general part, which is the other part than the connection in the connected members, the plastic behaviours are clarified. Therefore, the general part preferably fails before the connection. On the other hand, it is possible to design the connection by considering its nonlinearity from an economical point of view, or to let the connection fail before the general parts to secure safety of the whole structure. In these cases, connected section dominates performance of the whole structure, and the behaviours of the connection and the whole structure must be carefully studied after the connection is damaged. An improvement of verification technology such as numerical analysis considering nonlinearity of the connection can lead to a more rational and economical design of mixed structures than the current design.

6.3.2 Verification of section failure at connection

Safety of the connection must be verified according to the flow of forces, by assuming the force transfer mechanism to let sectional forces be surely transferred.

Major failure types of the connection in mixed structures are as follows.

- Bending failure: yield of main steel members in tension, or buckling of main steel members in compression or crush of concrete, at the connected section of a series connection type.
- Shear failure: shear failure of core concrete at the connection
- Shear transfer failure: shear slip failure at the connected section of a series connection type.
- Bond and anchor failure: bond and anchor failure of column in core concrete of the connection or main steel members of beams.
- Bearing failure: bearing failure of concrete contacting steel members

The above failure types are dominated by concrete, except bending failure, and a failure mode is brittle with small ductility and energy absorption. As there are various force transfer mechanisms in each structure, no common method to accurately estimate the failure strength has been established. Therefore, as for transferring sectional forces of axial forces, shear forces and bending moments at the connection of different types of members, steel members and concrete members are allowed to partly exceed the elastic limits in the connection, but it is designed to prevent compressive concrete crush. The maximum of the transferred force of transfer mechanism (transfer elements) should not reach the ultimate strength.

When multiple transfer elements resist sectional forces, a ratio of share among transfer elements should be properly evaluated and the limit value should be set to satisfy compatibility conditions between transfer elements. For example, when sectional forces are transferred by multiple transfer elements such as shear connectors, bearing and friction, sharing ratios of transfer elements are decided by proper analytical models, and then they are superposed to get the section strength. In this case as shown in Fig. 6.3.1, as the maximum transferred forces of transfer elements A and B do not always occur at the same displacement, this effect must be considered when a limit value is set for each transfer element. When the sharing ratio is uncertain, a verification should be done by considering only one major transfer element.

In verifying the connection, a distribution profile of transferred forces of transfer elements should be obtained based on the assumed force transfer mechanism. For example, when shear connectors are widely arranged, all the shear connectors do not equally work at the same time, and the distribution of transfer forces due to arrangement of shear connectors must be obtained by proper methods, as shown in Fig. 6.3.2.

In mixed structures, various connection methods of the different types of members are used, as described in 6.2. For example, Fig. 6.3.3 shows the bearing connection of a steel member and a PC member which are connected in the axial direction (series type). Fig. 6.3.4 shows the connection of a steel girder and a RC column with a right angle between member axes (perpendicular type). There are many transfer elements such as, shear connectors, bearing, friction, pre-stress, reinforced steel bars, and so on. The connection is verified for axial forces, shear forces and bending moment acting on the connection1).

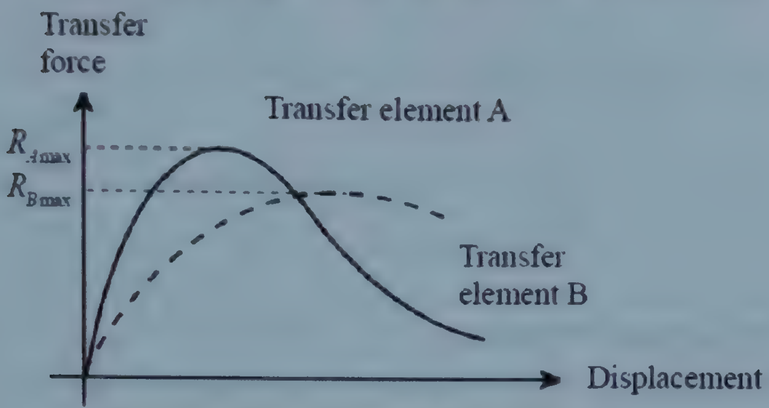


Fig.6.3.1 Superpose of strength of transfer elements

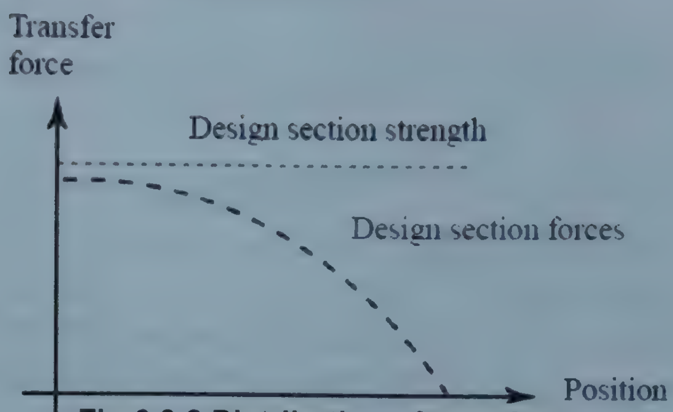


Fig.6.3.2 Distribution of transfer forces of transfer elements

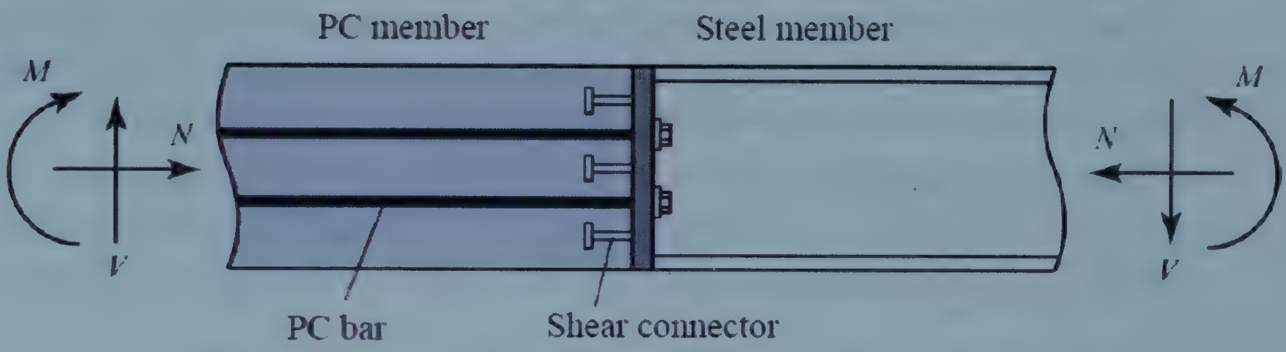


Fig.6.3.3 Bearing connection of steel member and PC member

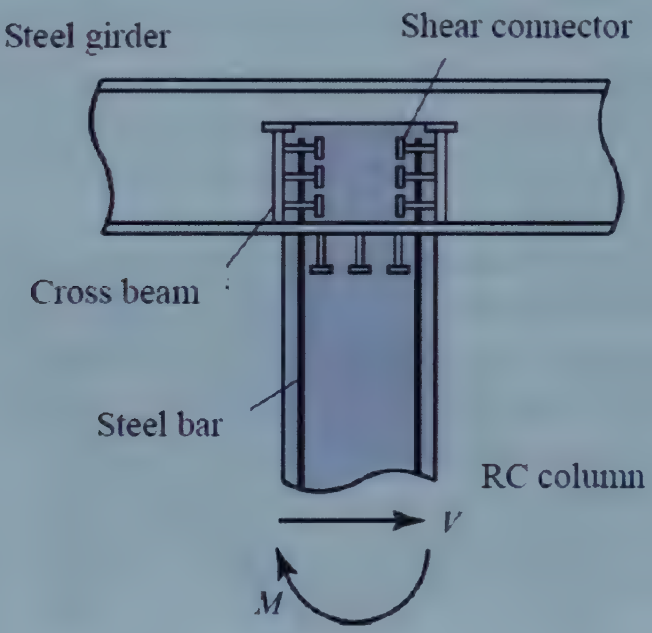


Fig.6.3.4 Connection of steel girder and RC pier

(1) Transfer of the axial forces

$$\gamma_i \cdot \frac{N_d}{N_{ud}} \leq 1.0 \qquad (6.3.1)$$

$$N_{ud} \,\square\, a_1 N_{sud} \,\square\, a_2 N_{bud} \,\square\, a_3 N_{oud} \qquad (6.3.2)$$

where,

N_d: Design axial force
N_{ud}: Axial strength (resistance by shear connectors, resistance by bearing force of steel and concrete)
N_{sud}: Share of strength by shear connectors
a_1: Share ratio of strength by shear connectors
N_{bud}: Share of strength by bearing of steel and concrete

$a2$: Share ratio of strength by bearing of steel and concrete
$Noud$: Share of strength by other transfer mechanism
$a3$: Share ratio of strength by other transfer mechanism
▯$_i$: Structure factor

$$N_{sud} = \sum_{i=1}^{n} V_{di} \qquad (6.3.3)$$

Provided that: $V_{di} < V_{sud}$

where,

Vd: transfer force due to shear force of a shear connector
$Vsud$: shear strength of a shear connector
n: number of shear connectors resisting axial force

Transfer force of each shear connector is obtained based on a distribution profile of transfer force for the assumed transfer mechanism. Therefore, the total sum of transfer force of individual shear connectors is the strength of the group of shear connectors, where the maximum transfer force in a shear connector among the group of shear connectors is smaller than the shear strength of a shear connector.

(2) Transfer of shear forces

$$\gamma_i \cdot \frac{V_d}{V_{ud}} \le 1.0 \qquad (6.3.4)$$

$$V_{ud} = b_1 \sum_{i=1}^{n} V_{di} + b_2 V_{oud} \qquad (6.3.5)$$

Provided that: $V_{di} < V_{sud}$

where,

Vd: Design shear force
Vud: Shear strength (resistance by shear connectors, resistance by friction)
$b1$: Share ratio of shear strength by shear connectors
$Voud$: Share of shear strength by other transfer mechanism
$b2$: Share ratio of shear strength by other transfer mechanism
Vdi: Transfer force due to shear of a shear connector
$Vsud$: Shear strength of a shear connector
n: Number of shear connectors resisting shear force
▯$_i$: Structure factor

Transfer force of each shear connector is obtained based on a distribution profile of transfer force for the assumed transfer mechanism.

(3) Transfer of bending moment

$$\gamma_i \cdot \frac{M_d}{M_{ud}} \le 1.0 \qquad (6.3.6)$$

$$Mud \square c_1Msud \square c_2Mbud \square c_3Moud \qquad (6.3.7)$$

where,

Md: Design bending moment
Mud: Bending strength (resistance by shear connectors, resistance by bearing of steel and concrete)
$Msud$: Share of bending moment by shear connectors
$c1$: Share ratio of bending moment by shear connectors
$Mbud$: Share of bending moment by bearing of steel and concrete

c2: Share ratio of bending moment by bearing of steel and concrete
Moud: Share of bending moment by other transfer mechanism
c3: Share ratio of bending moment by other transfer mechanism
▯*i*: Structure factor

When bending moment is resisted by a couple of forces, the verification method becomes the same as that of axial forces.

For the safety of the connection of members of different types and the general part of members, Standard Specifications for Hybrid Structures 2014$^{5)}$ specify that Eq. (6.3.8) should be satisfied. When the connection is designed so that it will not fail before the general part adjacent to the connection does, the part (*d1*) that fails first is defined as the general part. On the other hand, when the connection is designed so that it can fail before the general part adjacent to the connection does, the part (*d1*) that fails first is defined as the connection.

$$\alpha_m \frac{S_{d2}}{R_{d2}} \le \frac{S_{d1}}{R_{d1}} \qquad (6.3.7)$$

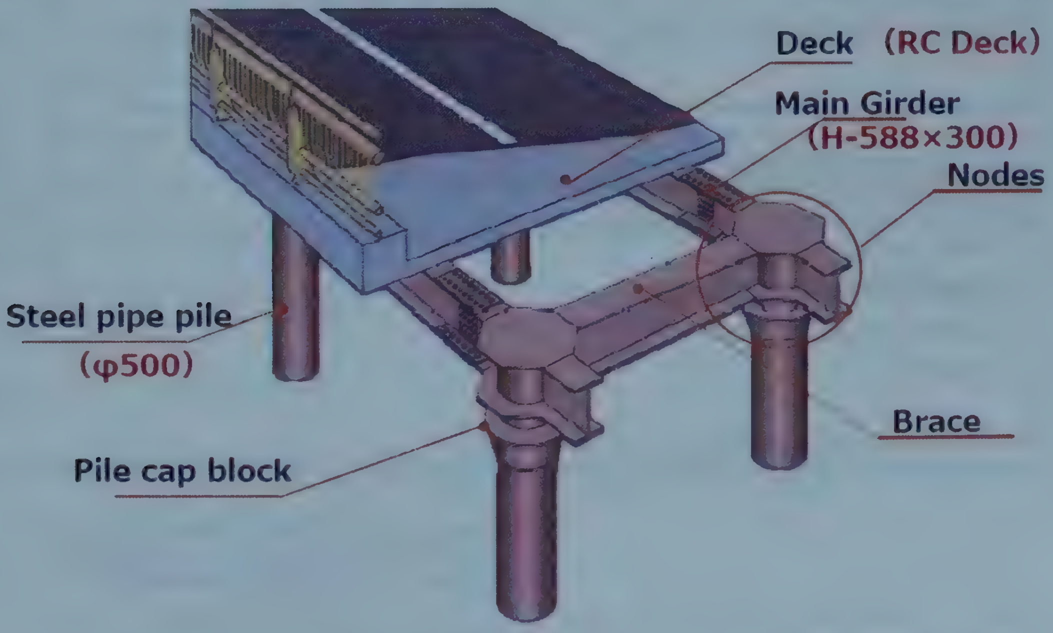
where,

Sd1, *Rd1*: Design sectional force at the first failure position, its design sectional strength
Sd2, *Rd2*: Design sectional force at a general position which does not fail first, its design sectional strength.
αm: Factor to control limit states of safety of the connection and general parts, 1.2 in general

[Source: Basic Theory and Design of Hybrid Structures -2019-, JSCE]

(5) Multipolar Three Dimensional Frame with RC Deck

1) Multipolar three dimensional frame with RC deck is prefabricated jetty ridge frame bridge consisting of steel pipe piles, H-shape girders, nodes and decks. Piles, main girders, and braces are rigidly connected. It has advantages in widening project in steep terrain.



(Source: https://www.jfe-civil.com/infra/metalroad/)

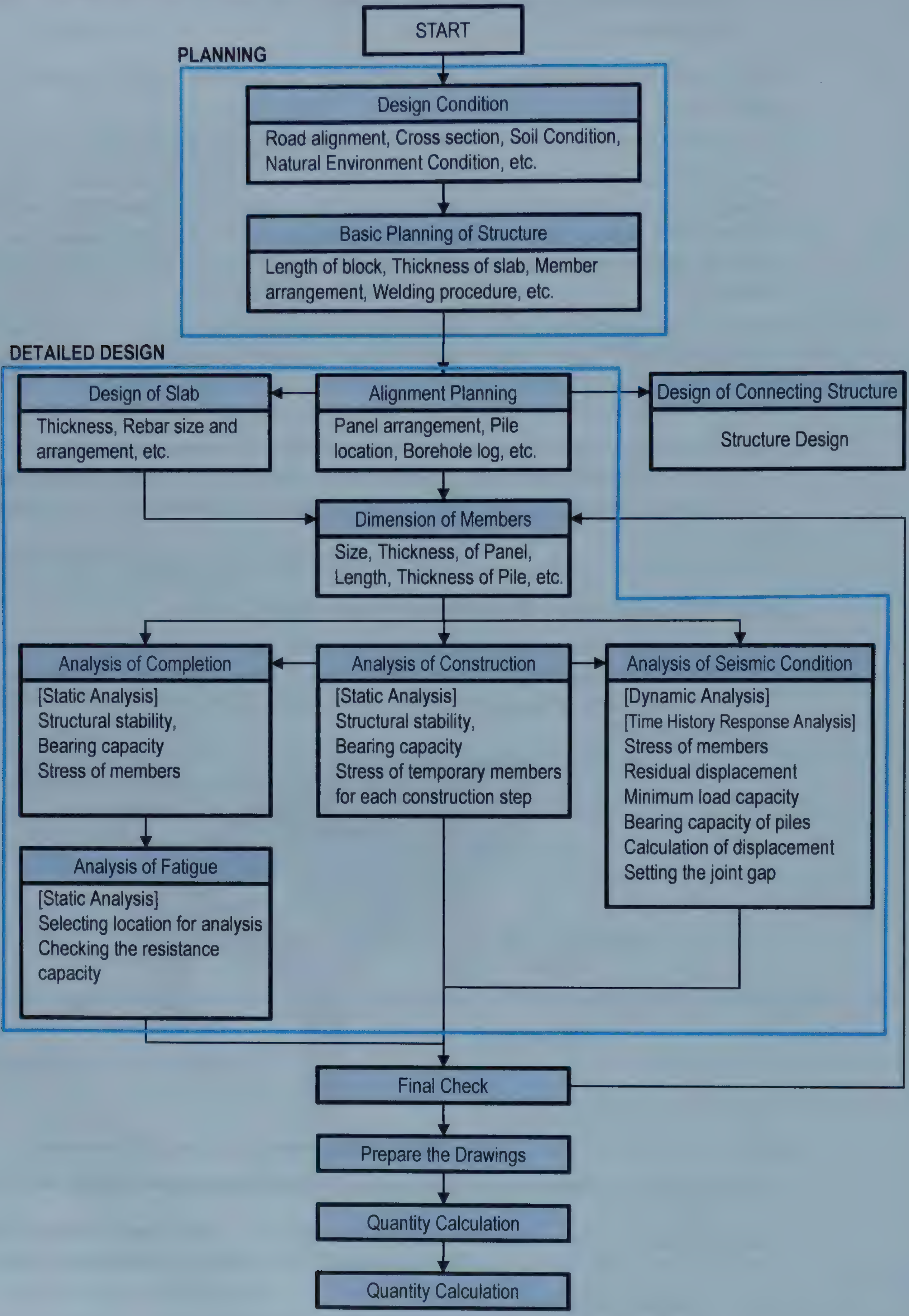
Figure 3-6: Structure of Multipolar Three Dimensional Frame with RC Deck

2) Multipolar three dimensional frame with RC deck has following advantages to ordinary cut and fill, cut & EPS (Expanded Polystyrol) fill method or ordinary bridge.

- High anti-seismic capacity
- High constructability
- Short construction period
- Less impact to the existing traffic

- Environmental friendly

3) Design procedure are described in the figure below


START
PLANNING
Design Condition
Road alignment, Cross section, Soil Condition, Natural Environment Condition, etc.
Basic Planning of Structure
Length of block, Thickness of slab, Member arrangement, Welding procedure, etc.
DETAILED DESIGN
Design of Slab
Thickness, Rebar size and arrangement, etc.
Alignment Planning
Panel arrangement, Pile location, Borehole log, etc.
Design of Connecting Structure
Structure Design
Dimension of Members
Size, Thickness, of Panel, Length, Thickness of Pile, etc.
Analysis of Completion
[Static Analysis]
Structural stability,
Bearing capacity
Stress of members
Analysis of Construction
[Static Analysis]
Structural stability,
Bearing capacity
Stress of temporary members for each construction step
Analysis of Seismic Condition
[Dynamic Analysis]
[Time History Response Analysis]
Stress of members
Residual displacement
Minimum load capacity
Bearing capacity of piles
Calculation of displacement
Setting the joint gap
Analysis of Fatigue
[Static Analysis]
Selecting location for analysis
Checking the resistance capacity
Final Check
Prepare the Drawings
Quantity Calculation
Quantity Calculation

(Source: https://www.jfe-civil.com/infra/metalroad/)

Figure 3-7: Design Procedure of Multipolar Three Dimensional Frame with RC Deck

4) There are three types of multipolar three dimensional frame with RC deck as follows,

- Widening type
- Mountain side cantilever type
- Independent type

5) Ordinary steel material are utilized as well as weathering steel. 400 to 500 N/mm^2 is applied for piles, main girders and braces.

3.2.3.5 Bearings, Expansion Joints, Railing, Wearing Coat and Appurtenances

(1) General

1) Miscellaneous members which support the bridges system are mainly attached to superstructure. They are always constructed while the superstructure is being constructed or just before/after. Therefore, applied types of miscellaneous members shall be selected and designed in details together with the superstructure's design procedure.

2) These miscellaneous members often have shorter durability than the service life of the bridge. It means that they shall be replaced periodically or when it is deteriorated during the service life of bridge. Therefore, accessibility to the member to inspect the condition is very important to maintain the service level of the bridge.

3) Damages to miscellaneous members are relatively small and does not affect the entire bridge system often. However, damages of bearings could cause superstructure drop and leave unexpectedly wide/big gap at the end of the superstructure. Damages of expansion joints is likely to leave gaps or damages the tyres of running vehicles. Damages of railings, including of crush barriers increase the risk of bridge user (drivers, passengers, and pedestrians), Damages of wearing coat leaves pot hole on the surface of the bridge which raises the risk of accident. Appurtenances, such as drainage system, lighting system, increases the risk of car accident and also speed of deterioration of other members. Damage on the paint on the members accelerated the deterioration of the members coated by the paint. Every damage on miscellaneous members shall not be left untreated. Therefore easiness of inspection and repair/replacement shall be taken into account in design.

4) Mountain bridges are often very far from maintenance office and not easy to inspect/repair/replace. Therefore it is not easy to maintain the required service level. Durability is the one of the key factors for selecting each items.

5) Bearings and devices to resist massive earthquake is described in details in Chapter 5.

(2) Bearings

1) There are fore IRC Codes for bearings, such as IRC:83 as follows,

- Part-I: Roller & Rocker Bearings, 2015
- Part-II: Elastomeric Bearings, 2015
- Part-III: Pot, Pot-Cum-PTFE, Pin and Metallic Guide Bearings, 2002
- Part-IV: Spherical and Cylindrical

2) Elastomeric bearings shall have top priority to be selected among above four types of bearing for long span bridges due to its high capacity to resist seismic force.

3) Elastomeric bearings are also suitable for small and medium bridges for its easy replacement and durability compared steel/metallic bearings due to improvement of its quality during these several decades.

4) The details of the detailed design of bearings shall refer to IRC:83, especially IRC:83 Part-II-2015 for elastomeric bearings.

(3) Expansion Joints

1) There is IRC Code for expansion joint to be referred to. It is IRC SP:69-2011. There are more than 10 types of expansion joint for comparison which are suitable for different amount of movement.

2) Durability and easiness of maintenance and replacement shall be given high priority in selection the types of expansion joints since mountain bridges are often far from maintenance office and not easy to inspect/repair/replace.

3) Expansion joints are very easy to be damaged severely when there are considerable amount of overloaded transport vehicles, such as truck, trailers. Therefore, to prevent severe damage of expansion joint, overloaded vehicles shall be prohibited and controlled on the highway/expressway. This issue shall be treated in detailed design stage to prepare the operation & maintenance manual.

(4) Railing, Protection barrier

1) Railing and Protection barrier is designed based on its material. If it is made of RCC, IRC 112-2011 shall be referred to, if it is made of steel, IRC:24-2010 shall be referred to.

(5) Wearing Coat

1) For wearing coat, IRC:107-2013 shall be referred to.

(6) Appurtenances

1) There are drainage facilities, kerb, approach slab, inspection platform, and etc. For drainage, IRC SP:42-2014 shall be referred to. For kerb and approach slab, IRC 112-2011 shall be referred to since it is normally made of RCC. For inspection platform, IRC:24-2010 shall be referred to.

3.2.3.6 Substructure

(1) General

1) Substructures are mainly divided into two types, such as abutments and piers. Abutments are usually located at the both ends of the bridge to sustain the superstructure as well as backfill earth behind the wall. Piers are usually located between the abutments to sustain the load transmitted from superstructure.

2) Mountain bridges has tendency to have substructure located at the middle of steep slope, within high velocity river flow, or with shallow bearing layer with fragile natural environment. The conditions of the site of mountain bridges are quite different than the site of the bridges in different terrain with flat terrain with deep baring layer.

3) If the route of the bridge which detailed design is to be executed is totally new, securing the access route to the location of substructures, specifically piers, shall be carefully studied to decrease the construction cost and mitigate the natural environment impact. This stage is the realistically the last chance to improve the construction planning including the temporary access route. Therefore, review shall be executed twice before the detailed design based on the study result of pervious study, such as feasibility study and after the detailed design based on the result of it.

4) The basic policy to reduce the construction cost and natural environment impact is smaller area of substructure and shorter length of temporary access route. To reduce the area of substructure, foundation shape, types and numbers of piles shall be carefully compared. To reduce the length of temporary access route, not only cut and fill road construction but also applying the temporary bridge/jetty shall be study.

5) There are IRC:78-2014 'Standard Specification and Code of Practice for Road

Bridges, Section VII Foundation and Substructure' to be referred for design methodology and structural details to be followed.

(2) Abutment

1) Abutments shall bear the load transmitted from superstructure and earth pressure behind the wall. Moreover, it shall resist to the flood flow in front of it if the abutment is located within the river section.

2) There are basically tree types of abutment which could be applied for mountain bridges on highways/expressway, 1st is wall type (both cantilever and counterfort), 2nd is box type, and the 3rd is the stud/sloping abutment. For small bridges, integral abutment could be one of the alternatives, but is shall be rare to be applied in highway/expressway project with numerous numbers of bridge comprised considerable part of the project.

3) To reduce the construction cost and natural environmental impact, area of abutment shall be kept as small as possible. To realize this policy, area of foundation shall be limited. Since the abutments of the mountain bridges are located in mountain or steep terrain, height of the abutment shall be minimized to reduce the area of foundation.

4) For wall type abutment, cantilever type is usually selected since the height of the wall is minimized. Counterfort type is only applied when there are constraints that limit the dead load of the abutment and the height of the wall are not able to be reduced.

5) Box type shall be avoided for mountain bridges due to its large size. However it can be applied to in relatively gentile slope with local road crossing underneath. The local road could be placed in the box section to reduce the length of superstructure for cost reduction.

6) Stud/sloping abutment has advantage in steep terrain without the risk of scouring. These abutments are usually low in height and increase the superstructure length. However, without having river/flood flow crossing in front of it and small earth pressure from behind of it, it could be cheaply installed, safely maintained, and reduce life cycle cost of the bridge.

7) Abutments shall not be located within the influential area of land slide nor the river/flood flow with debris flow. However, if the abutment is indispensable in these natural disaster risk area mentioned above for the bridge system, suitable analysis of the scale of the landslide/debris flow and install countermeasure facilities. Specifically protection structure just by the upstream side of the pier for debris flow shall be installed.

8) The design methodology and structural details are described in IRC:78-2014 which shall be referred to.

(3) Pier

1) General

i) Piers shall bear the load transited from superstructure. It is normally free from earth pressure but it shall resist against the river/flood flow if it is located within the flow.

ii) There are basically three types of piers which could be applied for mountain bridges on highways/expressway, 1st is wall type, 2nd is column type, and the 3rd is rigid frame (multi-column) type. There shall be steel piers as an alternative, but it is rarely applied for the mountain bridges considering the design, construction, and procurement conditions. T-shaped typed with cantilever girder as a pier cap, is also shall be an alternative. However, this type of pier is included in wall type or column type depending on the shape of vertical cross section at under the cantilever girder.

iii) For small bridges, pile cap type pier could be one of the alternatives due to easy construction. However, the countermeasure of special treatment shall be applied for long free pile section. If the pier is located within the crossing river/flood section countermeasure to souring requires special attention. Moreover, durability of free pile section shall be considered in the detailed design and protective maintenance method shall be applied to retard the deterioration progress since pile are impossible to replace during the service life of bridge.

iv) Piers shall not be located within the influential area of land slide nor the river/flood flow with debris flow. However, if the pier is indispensable in these natural disaster risk area mentioned above for the bridge system, suitable analysis of the scale of the landslide/debris flow and install countermeasure facilities. Specifically protection structure just by the upstream side of the pier for debris flow shall be installed.

2) Wall Type

i) Wall types are defined as ‘When the length of solid pier is more than four times its thickness’ in IRC:78-2014.

ii) Methodologies and structural details demonstrated in IRC: 78-2014, and IRC-112-2011 which shall be referred to.

iii) Wall type has advantage to resist to shear force in transverse direction transmitted to pier wall by seismic force. However, for cross sectional landscape, it interrupts the view and has disadvantage in aesthetic point of view. Therefore high pier shall not have very flat cross section to improve the appearance.

iv) Wall type has advantage for pier in river/flood flow because the thickness in longitudinal direction can be minimized. It could resist against scouring due to its cross sectional shape.

3) Column Type

i) Column types could be defined as ‘When the length of solid pier is up to four times its thickness’ judging from the definition of ‘Wall Type’ in IRC:78-2014.

ii) Methodologies and structural details demonstrated in IRC: 78-2014, and IRC-112-2011 which shall be referred to.

iii) It has advantaged in aesthetic point of view especially from distance, because it does not interrupt the view both transverse and longitudinal (cross section) direction.

iv) Bridges with very wide variety of height of piers or very unsymmetrical span arrangement with continuous superstructure or superstructure and substructure connected rigidly, reacts to massive earthquakes very complicated manner. In these analysis, column type reacts flexibly than wall type. However, it shall be verified by dynamic analysis. This is an important advantage for major mountain bridges for highway/expressway.

v) If the height is more than 20m, the cross section shall be compared with solid and hollow to reduce the quantity and dead load of the pier. The details of the high piers is described in clause below.

4) Rigid Frame Type

i) Rigid fame type is generally applied when there is constraints of placing structure at the pier location, such as crossing river/canal, highway/road, or railway. When these crossing objects are crossed with the bridge with keen angle, there shall be wide clearance underneath of bearings. Therefore, columns of rigid frame is placed outside of the clearance, and beam of the rigid frame supports the bearings and superstructure.

ii) However, the construction cost is much higher than wall type and column type piers. Therefore, it shall be strongly avoided by shifting the alignment of rearranging the span arrangement in the mountain region with relatively less restriction of placing piers.

iii) One more opportunity for applying the rigid frame type is replacing very wide wall type pier with multi column. This alternative shall be done in detailed design calculation, and the cost

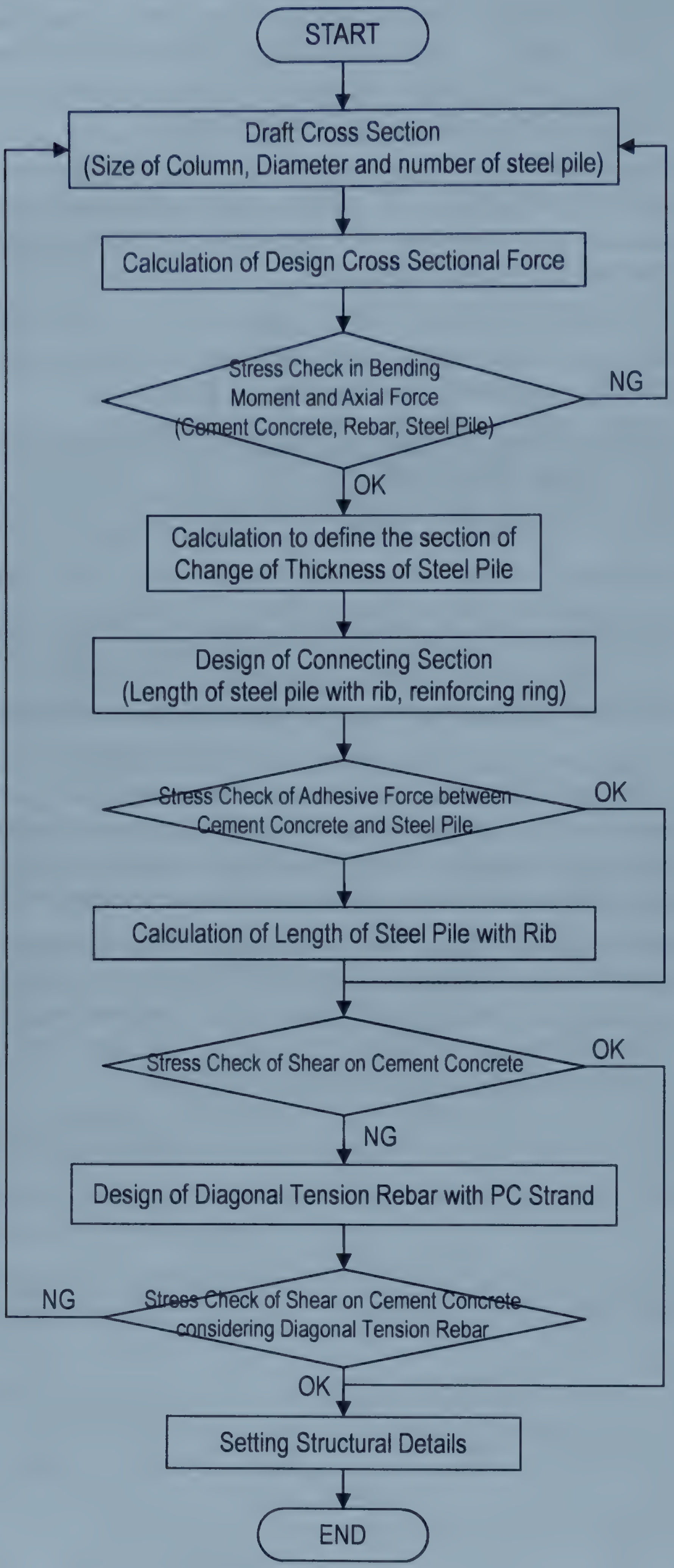
iv) Methodologies and structural details demonstrated in IRC: 78-2014, and IRC-112-2011 which shall be referred to.

5) High/Very High RCC Piers

i) Mountain bridges crossing deep valley with easy substructure construction condition require high/very high RCC piers. The world highest pier is over 300m of structural height. However, piers with its height 30m or more are generally considered high piers, and height with 100m or more as very high piers. This clause features the required specific considerations for high and very high piers.

ii) High/very high piers have enormous weight and its foundation, footing, and bottom end of the column shall bear massive vertical force. Their cross section shall bear the huge compression stress accordingly. Therefore, high performance cement concrete shall be studied for its application. The methodology and specification of high performance is described in IRC:112-2011 and relevant ISs.

iii) Its high height and massive dead load derives very large horizontal force, specifically by seismic and wind force. Therefore, hollow cross section and composite cross section with steel pipe are applied to reduce the dead load of the column specifically at the upper section. Design procedure of composite high piers with steel pipes are shown below,


START
Draft Cross Section
(Size of Column, Diameter and number of steel pile)
Calculation of Design Cross Sectional Force
Stress Check in Bending Moment and Axial Force (Cement Concrete, Rebar, Steel Pile)
NG
OK
Calculation to define the section of Change of Thickness of Steel Pile
Design of Connecting Section (Length of steel pile with rib, reinforcing ring)
Stress Check of Adhesive Force between Cement Concrete and Steel Pile
OK
Calculation of Length of Steel Pile with Rib
Stress Check of Shear on Cement Concrete
OK
NG
Design of Diagonal Tension Rebar with PC Strand
NG
Stress Check of Shear on Cement Concrete considering Diagonal Tension Rebar
OK
Setting Structural Details
END

(Source: Design Standard of NEXCO Japan, Part II, Bridge Construction)

Figure 3-8: Design Procedure of Composite High Piers with Steel Pipes

iv) Methodology and structural details of composite pier with steel piers are extracted from Design Standard of NEXCO Japan, Part II, Bridge Construction

Extracted from Design Standard of NEXCO Japan, Part II, Bridge Construction, NEXCO

Chapter 5 Substructure

4. Pier

4-4 Composite Pier of Steel Pile and RCC

4-4-1 Scope

This subchapter is applied for the design of composite pier of steel pile and cement concrete utilizing PSC strand as lateral tie.

4-4-2 General

(1) This structure shall be designed as composite structure of steel pipe and cement concrete and that it has a structural characteristics that PSC strand is used as lateral tie.
(2) This structure shall be designed against design seismic force considering the structural characteristics of composite pier of steel pile and cement concrete.

4-4-3 Typical Cross Section

(1) Typical cross section of this structure shall place vertical rebars near the external surface and steel pile internally. PSC strands are used as lateral ties and rapped around the vertical rebars
(2) Vertical rebars shall be placed near the external surface.
(3) Arrangement of steel pile
  1) Steel piles shall be arranged in matrix form. Steel piles shall not be arranged irregularly nor with numerous numbers.
  2) Diameter and thickness shall be set within the procurable products and optimised economically considering transportation condition and prevention of local deformation in construction stage.

4-4-4 Design against Bending Moment and Axial Force

(1) Design can be done with the assumption that steel pile and cement concrete are uniform structure in range of elasticity.
(2) Maximum adhesive stress on outer surface of steel pile between steel pile and cement concrete shall be less than (or equal to) permissive adhesive stress.

4-4-5 Design against Shear

(1) Shear force shall be checked that it is bearable at the several appropriate cross sections
(2) Design PSC strand
  - Design of PSC strand against seismic force caused by level 1 earthquake shall be done considering share of shear before the occurrence of diagonal crack born by steel pile and cement concrete.
  - Design of PSC strand against seismic force caused by level 2 earthquake shall be done in accordance with 4-4-6 below.

4-4-6 Design against Seismic Force (Level 2 Earthquake)

(1) Basic policy
Methodology and details of design of this structure against seismic force caused by level 2 earthquake is basically the same as those of RCC pier except the clause specifically mention in this section, 4-4-6.
(2) Judgement of Collapse
  1) Composite pier of steel pipe and RCC shall be collapsed as bending fracture.
  2) Collapse of composite pier of steel pipe and RCC hall be judged evaluating ultimate horizontal bearing capacity and shear capacity appropriately.
(3) Shear Capacity
Shear Capacity of composite pier of steel pipe and RCC shall be calculated evaluating share of shear of RCC, steel pipe and PSC strand appropriately.

(4) Relation between Bending Moment and Curvature
Relation between bending moment and curvature for nonlinear analysis shall be calculated evaluating adhesive property of steel pipe and RCC appropriately.

(5) Checking Reaction Rotation Angle at Plastic Hinge
Maximum reaction rotation angle shall be less than (or equal to) allowable rotation angle.

(6) Check against Shear Force
Check against shear force shall be checked that the maximum shear force at each member is less than (or equal to) shear capacity calculated at (3) above.

4-4-7 Structural Details

(1) Bending Radius of PSC strand
Bending radius of PSC strand at the corner of cross section shall be set appropriately considering the diameter of PSC strand and workability of bending at site.

(2) Cover of steel piles and Gap between steel piles
1) Cover of steel piles shall be secured appropriately considering arrangement of other steel members (rebar and PSC strand) and filling easiness of cement concrete (maximum size of aggregate).
2) Gap between steel piles shall be secured appropriately considering workability at the site.

(3) Vertical Rebar
Cross section of steel piles can be included in calculating minimum rebar sectional area.

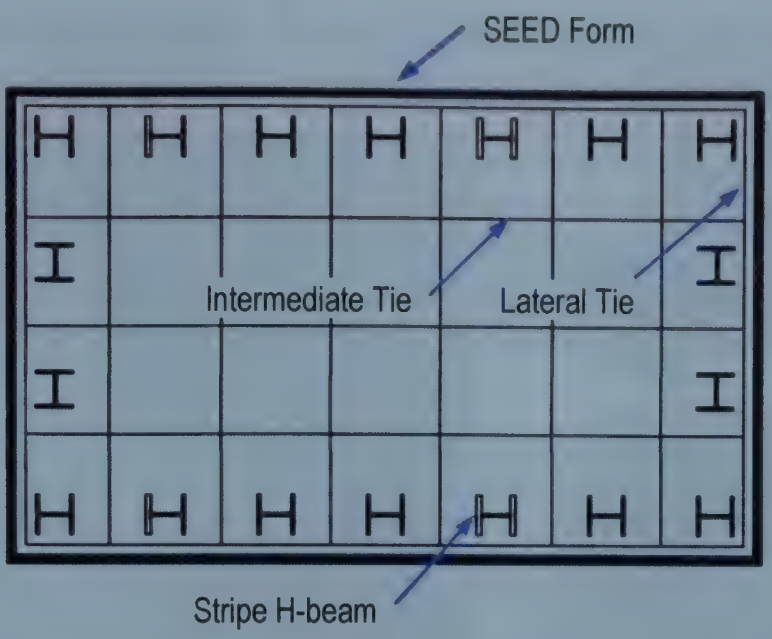
(4) PSC Strand
Minimum number of PSC stand at the base of pier shall be calculated evaluating plasticity of composite pier of steel pipe and RCC appropriately. Moreover, diameter and interval of PSC strand used as lateral ties shall be fixed considering the workability to set in spiral form and filling easiness of cement concrete (maximum size of aggregate).

(5) Anchoring of Steel Pipes
1) Steel pipes shall be anchored firmly at the appropriate location in order to unite steel pipes and cement concrete.
2) Local stress such as split failure around the external surface of steel pipes shall be carefully considered at the anchoring sections of the steel pipes.
3) Steel pipes shall be reinforced at the anchoring sections in order to transfer the forces smoothly, prevent sudden change of sectional rigidity, prevent local buckling of steel pipes, and slipping out of steel pipes after large deformation of steel pipes.

(6) Joint Structure of PSC Strand
Joint of PSC strands shall have the structure which transfer the strength of PSC strand firmly.

(Source: Design Standard of NEXCO Japan, Part II, Bridge Construction)

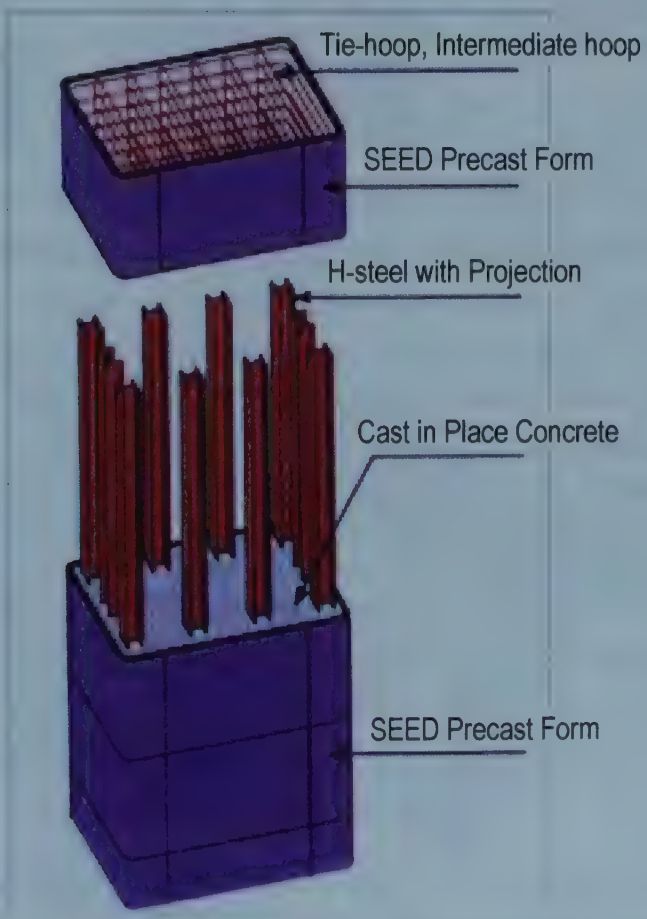
v) In other cases steel H-beam can be utilize to simplify the re-bar arrangement of the column since the tension stress derived by bending moment at the bottom end of the column tends to be considerably large and there will be very large amount of large diameter shall be arranged. Applying the H-beam can simplify the re-bar arrangement considerably as a sample below. (REED Method)



(Source: Design and Construction Manual of REED Method)

Figure 3-9: An Example Cross Section of Composite High Pier with Steel H-beam

vi) This construction method utilizes not only the steel H-beam but also the self-climbing steel form for improving the constructability of casting the concrete at site. The form will be sustained by H-bean and do not require other support. Moreover, it does not require scaffoldings either. This cannot be realize with ordinal re-bars. Applying this construction method, the construction load requires special consideration for temporary situation. The details are described in Chapter 4 below.



(Source: https://www.maeda.co.jp/tech/all/td0014.html)

Figure 3-10: Typical Structure of Composite High Pier with Steel H-beam

vii) Composite pier could be rigidly connected to superstructure and form rigid frame bridge. Superstructure could be either PSC structure of Steel structure. It also can be a composite structure demonstrated in 3.2.3.4. Superstructure (4) 2). If the superstructure is steel plate girder, it forms composite integrated (rigid framed) bridge demonstrated in Superstructure (4) 3). Outline of the composite integrated (rigid framed) bridge with composite pier (REED Method) is shown in figures below. (RI-Bridge Method; Rapid Integrated Bridge Method)

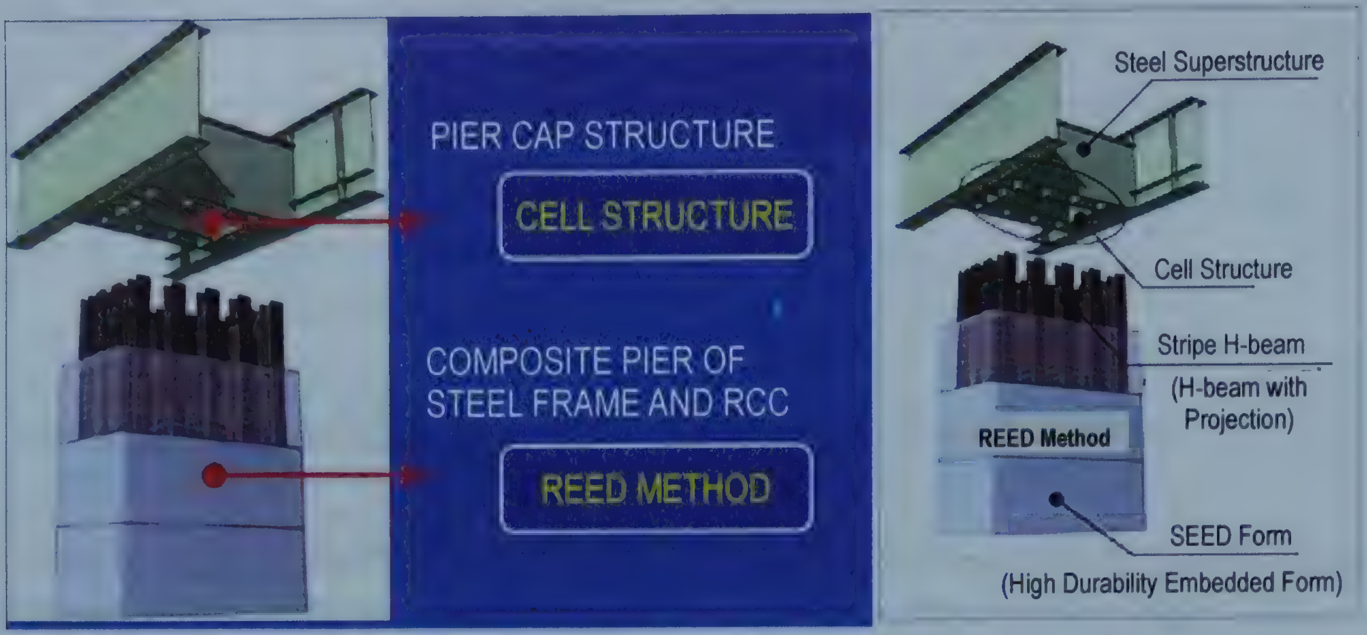


Figure 3-11: Image of Composite Integrated (Rigid Framed) Bridge with Composite Pier (REED Method) [RI-Bridge Method]

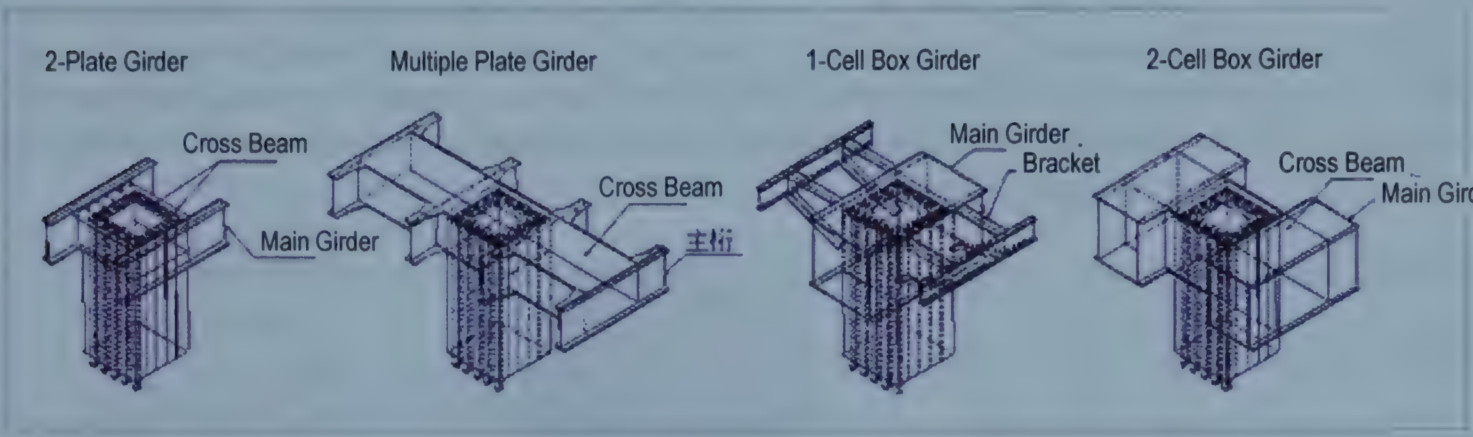
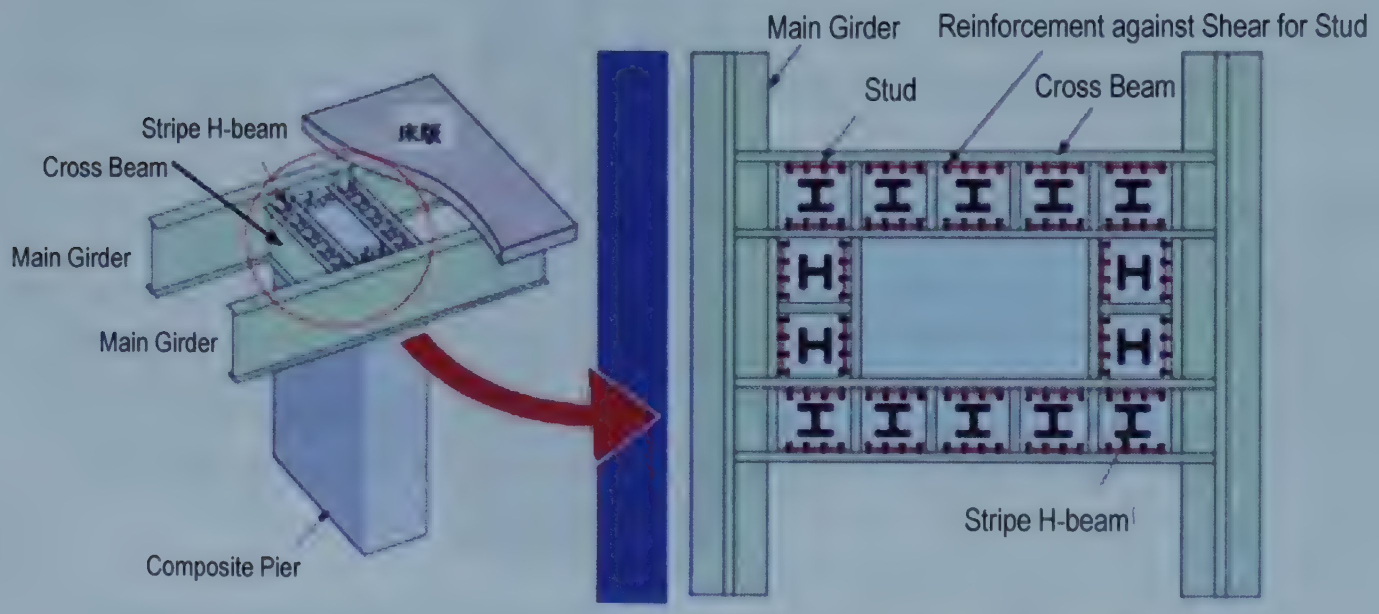


Figure 3-12: Variation of Connection by RI-Bridge Method



Plan Cross Section of Pier Cap of RI-Bridge Method

(Source: https://www.maeda.co.jp/tech/all/td0015.html)

Figure 3-13: Cross Section of Pier Cap of RI-Bridge Method

viii) Methodology and structural details of composite column are also described in IRC:22-2015 to be referred to.

3.2.3.7 Foundations

(1) General

1) Foundation shall bear the all the load transmitted from structures and members above and maintain the stability all through the service life of the bridge. It shall bear the vertical force of the entire bridge system, horizontal force from external environment, and moments derived by various sources. Therefore, foundation shall not sink, slide nor rotate by the loads or combination of those load act on the bridge system

2) Bearing layers for mountain bridges have tendency that it is located in shallow level of under the ground surface level. Therefore, spread foundation and caisson foundation has advantage of being applied. Though the bearing layer is relatively deep, there exist gravel of large size with its diameter larger than 10mm. The existence of gravel larger than 10mm of its diameter restricts construction of the types of piles and the most of pre-fabricated piles nor cast-in-situ piles are not able to be constructed. The remaining types of foundation is spread foundation, caisson foundation and Deep foundation.

3) IRC Codes for foundation is IRC:78-2014. Methodologies and structural details are described in it which shall be referred to.

(2) Spread Foundation

1) Spread Foundation on Flat Bearing Layer

i) Spread foundation is applied basically when the bearing layer is located at the depth shallower than 5m. If the bearing layer is located deeper than 5m, other types of foundation have more advantages both economically and technically.

ii) If the bearing layer are not forming slope, then the procedure and the details of the spread foundation are as same as those of spread foundation for bridges in other terrain. Methodologies and structural details shall be referred to IRC:78-2014. Since it is usually RCC structure, IRC:122-2011 shall also be referred to.

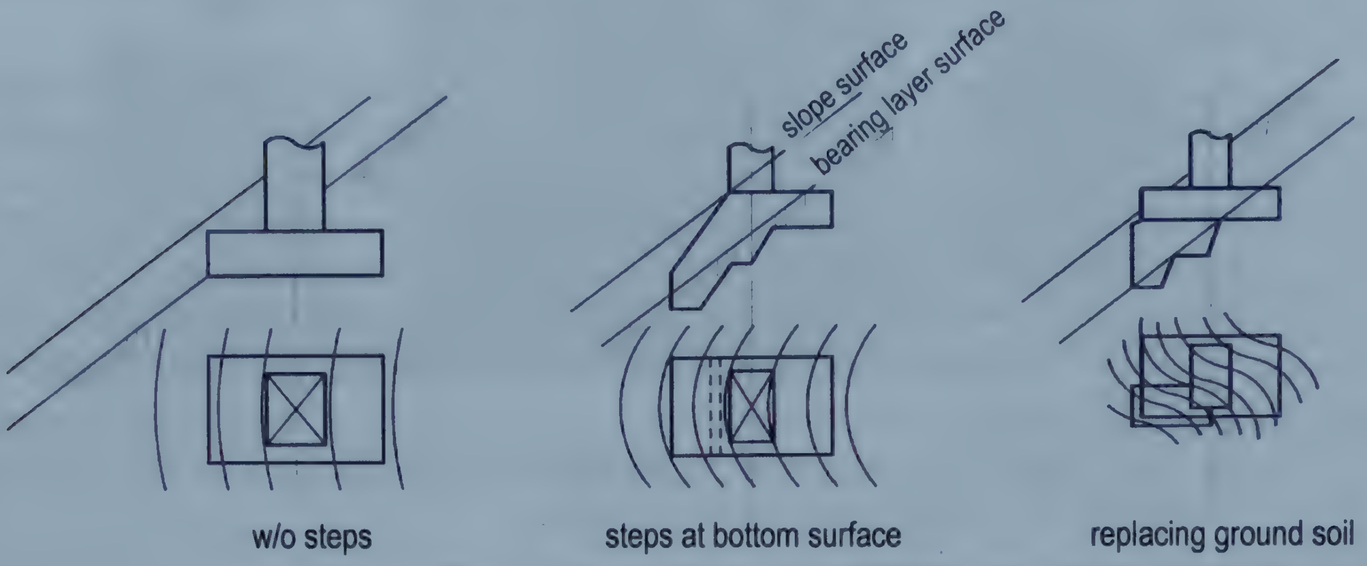
iii) However, bearing layer in mountainous region tends to be uneven with bump/hollow or forming spherical surface concavely or convexly. Flat bottom surface of spread foundation ends up not being touched to the bearing layer. Lack of bearing capacity of foundation could be occurred in this case. This fact can only be found by several boring survey within same foundation area. If the previous study result indicates the unevenness of the bearing layer, which is applicable in most of the cases, more than two boring investigation shall be executed for each foundation of substructure. If the results of the boring investigation indicates the complicated uneven surface of the baring layer and the foundation area is large, maximum number of the boring investigation necessary is five, which are four in each corner and one at the centre.

2) Spread Foundation on inclined Bearing Layer

i) If the spread foundation is located on inclined bearing layer, direction and degree of inclination shall be defined in the detailed design. And the bottom surface shall be firmly contacted to bearing layer to secure the bearing capacity of the foundation.

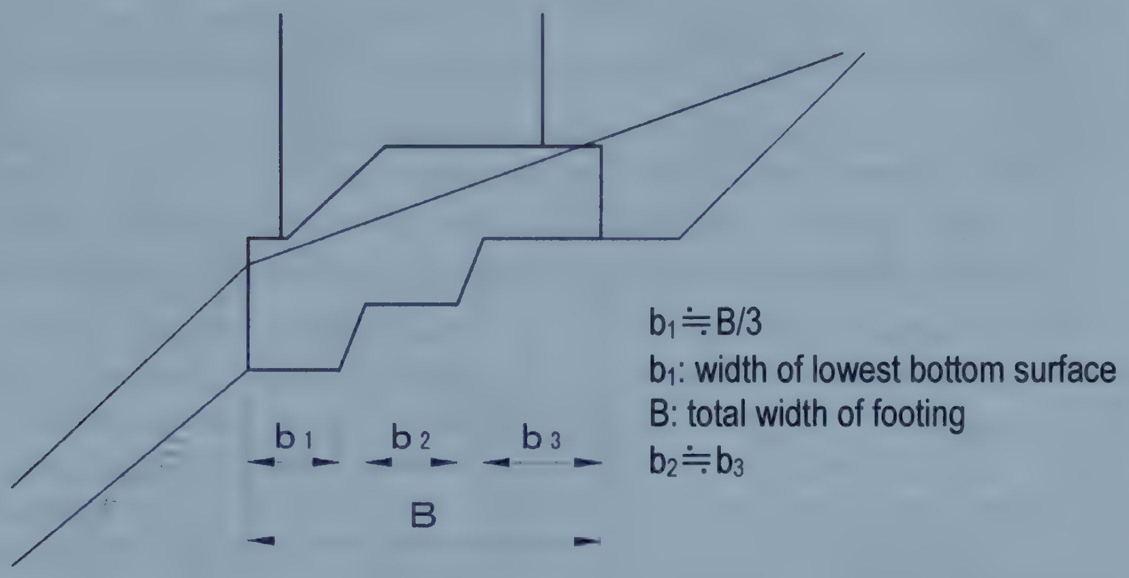
ii) It is very often that the bearing layer of mountain bridges are inclined. Direction and degree of inclination is only defined by multiple boring investigations within the foundation. If the results of the boring investigation indicates the complicated uneven surface of the baring layer and the foundation area is large, maximum number of the boring investigation necessary is five, which are four in each corner and one at the centre.

iii) If the slope of the bearing layer is inclined steeply, special countermeasure, such as steps or replacing the ground soil, shall be taken. Footing with steps on the bottom surface has advantage when the direction of the slope is in parallel with the surface of wall of abutment. Replacing the ground soil is often applied when the direction of the slope is diagonal to the wall of abutment (refer to the Figure below). Typical example of spread foundation with steps at the bottom surface of footing is also shown below.



(Source: Design Standard of NEXCO Japan, Part II, Bridge Construction)

Figure 3-14: Countermeasures for Spread Foundation on the Steep Slope


$b_1 \doteqdot B/3$
b_1: width of lowest bottom surface
B: total width of footing
$b_2 \doteqdot b_3$
b 1
b 2
b 3
B

(Source: Design Standard of NEXCO Japan, Part II, Bridge Construction)

Figure 3-15: Typical Spread Foundation with Steps at the Bottom Surface of Footing

(3) Pile Foundation

1) General

i) Pile foundation is applied basically when the bearing layer is located at the depth deeper than 5m. If the bearing layer is located shallower than 5m, spread foundation have more advantages both economically and technically. Mountain bridges tends to have the bearing layer shallow. Therefore, piles also tends to be shorter than for the bridges in flat terrain.

ii) Pile foundation is compared with caisson/well foundation. If the sub-soil is easy for excavation, pile foundation have more advantage than caisson/well foundation in general. However, sub-soil condition shall be investigated, analysed, and studied carefully in comparison study, so as not to apply the type of pile or construction method of pile which are not suitable, and required revision in construction stage. Mountain revers often have big size gravel and has sub-soil condition unsuitable for pile construction.

iii) Physical characteristics of sub-soil strata are basic condition of foundation structural calculation. It is also the basis of the condition to judge the constructability, for instance, the depth/hardness of the baring layer, the size of the gravel of intermediate layers, permeability, etc. regulates the types of piles. Both steel and concrete pre-fabricated piles have limited condition of contraction compared to cast-in-situ piles. It is not often that the pre-fabricated piles are applied in mountainous and steep terrain, not only because of the physical conditions of sus-soil strata, but also of the high construction cost. Pre-fabricated piles shall be planned considerable number to reduce the cost per pile, which means the sites of bridges require similar physical conditions of sub-soil strata. However, the conditions are often unique by sites of the bridges, and often requires custom-built foundation. Therefore, it is rare that pre-fabricated piles have advantage economically.

2) Types of pile foundation

i) There are two groups of types of pile foundations. One is pre-fabricated piles and the other is cast-in-situ piles.

ii) Both groups are divided into several categories by construction method. Pre-fabricated piles are divided into four categories, such as driven pile, inter-excavation piles, pre-boring piles, and steel piles filed with soil cement. Cast-in-site pile are also divided into four categories, such as all-casing (driven cast-in-situ), reverse circulation, earth-drill, and rotary driven casing. Each categories have its advantages and disadvantages, and the applicable conditions of sub-soil strata.

Table 3-12: Applicable Conditions of Sub-soil Strata and Constructability by Categories of Pile Foundation

Type of Foundation				Spread	Pile															Deep		Caisson		Steel pile sheet pile	Underground continuous diaphragm
					Driven			Inter-excavation						Steel pile soil cement	Pre-boring	Bored			Rotary						
						Steel pile		PHC, SC			Steel Pile														
Application Condition					PHC, SC	Driven	Vibrating Hummer	Driven to bearing layer	Agitating and jetting	Concrete casting	Driven to bearing layer	Agitating and jetting	Concrete casting			All-casing	Reverse circulation	Earth drill		Multiple with pile cap	Single	Pneumatic	Open		
Subsoil Condition	Subsoil condition upper than bearing		Very Soft Layer	×	○	○	○	○	○	○	○	○	○	○	○	×	○	○	○	×	×	○	△	○	○
			Intermediate very hard layer	×	△	△	△	○	○	○	○	○	○	○	○	△	○	×	○	○	○	○	△	△	○
		Gravel diameter	~50mm	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
			50~100mm	×	△	△	△	△	△	△	△	△	△	○	○	△	×	○	○	○	○	○	○	△	△
			100~500mm	×	×	×	×	×	×	×	×	×	×	×	×	△	×	×	×	○	○	○	△	×	△
			Liquefaction Layer	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	×	×	○	○	○	○
	Condition of bearing layer	Depth	~5m	○	×	×	×	×	×	×	×	×	×	×	×	×	×	×	×	○	×	×	×	×	×
			5~15m	△	○	○	○	○	○	○	○	○	○	○	○	○	△	○	○	○	○	○	○	△	△
			15~25m	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
			25~40m	×	○	○	○	○	○	○	○	○	○	○	○	○	○	△	○	△	△	○	○	○	○
			40~60m	×	△	○	○	△	△	△	○	○	○	○	○	△	○	×	○	×	×	△	○	○	○
			60m~	×	×	△	△	×	×	×	×	×	×	△	△	×	△	×	○	×	×	×	△	△	△
		Type of Soil	Sand, Gravel	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
			Clay	○	○	○	○	○	△	×	○	△	×	△	△	○	○	○	△	○	○	△	△	○	○
			Soft rock, hardpan	○	×	○	△	○	△	×	○	△	×	△	△	○	○	○	△	○	○	○	○	○	○
			Hard rock	○	×	×	×	×	×	×	×	×	×	×	×	△	△	△	×	○	○	△	×	×	△
			Steep slope, bumpy	△	△	△	△	△	△	△	△	△	△	△	△	○	○	○	○	○	○	△	×	○	○
	Underground water		Near ground surface	△	○	○	○	○	○	○	○	○	○	○	○	△	△	△	○	△	△	○	○	○	△
			Large volume of spring	△	○	○	○	○	○	○	○	○	○	△	△	△	△	△	○	×	×	○	○	○	△
			Confined above surface (2m~)	×	○	○	○	×	×	×	×	×	×	×	×	×	×	×	○	×	×	△	△	○	×
			Flow (3 m/min ~)	×	○	○	○	○	×	×	○	×	×	×	×	×	×	×	○	×	×	○	△	○	×
Bearing Type			Bearing pile	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
			Friction pile	×	○	○	○	×	×	×	×	×	×	○	×	○	○	○	×	×	×	×	×	×	×
Construction condition	Depth of Water		~5m	△	○	○	○	△	△	△	△	△	△	×	×	×	×	×	○	×	×	△	△	○	×
			5m~	×	△	○	○	△	△	△	△	△	△	×	×	×	×	×	○	×	×	△	△	○	×
			Small yard	○	△	△	△	△	△	△	△	△	△	△	△	△	△	△	△	○	○	△	△	×	
			Diagonal pile	×	○	○	○	×	×	×	×	×	×	×	×	×	×	×	○	×	×	×	×	×	×
			Poisonous gas	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	×	×	×	○	○	○
	Surrounding environment		Vibration, noise countermeasure	○	×	×	△	△	○	○	△	○	○	○	○	○	○	○	○	○	○	○	○	×	○
			Existing structure adjacent	○	×	△	△	△	○	○	△	○	○	○	○	○	○	○	○	△	△	△	△	△	○

note) ○: Applicable, △: Require Consideration, ×: Not applicable

[Source: Specification for Highway Bridges of Japan (Part IV Substructure)]

3) Pile Foundation on Flat Bearing Layer

i) If pile foundation are as same as those of pile foundation for bridges in other terrain. Methodologies and structural details shall be referred to IRC:78-2014. If it is RCC structure, IRC:122-2011 shall also be referred to. If it is steel structure, IRC:24-2010 shall also be referred to.

ii) However, bearing layer in mountainous region tends to be uneven with bump/hollow or forming spherical surface concavely or convexly. Length of piles which forms single foundation system of a substructure could differ accordingly or requires deep penetration depth into bearing layer excessively to unite the length of each pile. If the applied category of piles are one of the prefabricated piles, some of the piles does not reach to bearing layer and some would be damages by penetrating to the bearing layer excessively. Lack of bearing capacity of foundation could be occurred in this case. This fact can only be found by several boring survey within same foundation area. If the previous study result indicates the unevenness of the bearing layer, which is applicable in most of the cases, more than two boring investigation shall be executed for each foundation of substructure. If the results of the boring investigation indicates the complicated uneven surface of the baring layer and the foundation area is large with many piles, maximum number of the boring investigation necessary is five, which are four in each corner and one at the centre.

4) Pile Foundation on inclined Bearing Layer

i) If the pile foundation is located on inclined bearing layer, direction and degree of inclination shall be defined in the detailed design. And the bottom end shall be firmly contacted and penetrated into bearing layer to secure the bearing capacity of the foundation.

ii) It is very often that the bearing layer of mountain bridges are inclined. Direction and degree of inclination is only defined by multiple boring investigations within the foundation. If the results of the boring investigation indicates the complicated uneven surface of the baring layer and the foundation area is large, maximum number of the boring investigation necessary is five, which are four in each corner and one at the centre.

iii) If the slope of the bearing layer is inclined steeply, special countermeasure, such as different length of piles with in on foundation system of a substructure shall be taken. This method could be applied much more easily for cast-in-situ piles than pre-fabricated piles.

iv) Methodologies and structural details shall be referred to IRC:78-2014. It is RCC structure, IRC:122-2011 shall also be referred to. If it is steel structure, IRC:24-2010 shall also be referred to.

5) Cast-in-situ piles

i) Cast-in-site pile are divided into four categories, such as all-casing (driven cast-in-situ), reverse circulation, earth-drill, and rotary driven casing. Construction procedure of all methods are firstly excavate the sub-soil according to the design results, next set re-bar cage and cast concrete at last.

ii) All-casing (driven cast-in-situ) method uses steel casing for the guide in excavation. Usually it is jacked into the sub-soil by pressure or vibration. The casing is pulled out on casting the concrete to the ground. This method rarely have collapse of pile wall. Therefore it has advantage in fragile soil stratum, such as sand, or silt sand with/without small gravel. If guide casing can crack the gravel, it can excavate the stratum with quite large gravel. However, the driven machine for steel casing is quite large and requires large construction yard. It is one of the disadvantage for mountainous bridges.

iii) Reverse circulation methods excavate with water which contains bentonite with pressure. Bentonite and pressure holds the pile wall until the concreate casting, Therefore, sub-soil strata requires permissibility so not to lose the pressure. Clayey and silty strata is applicable for this method. Since it does not require long casing and drilling bit receives the help of circulation of the water, it is theoretically excavatable more than 100m long piles. However it is not applicable to apply for sandy stratum since it lose the water pressure.

iv) Earth-drill method excavates with bucket auger. The shaft is filled with pressured water and holds the pile wall. The water pressure is less than reverse circulation method. Therefore the sub-soil strata requires the permissibility but not as much as the reverse circulation method. However, it is difficult to hold the pile wall for more than 25m and longer length than 40m shall not be applied.

v) Rotary driven casing method is uses the steel casing for guide of the excavation as same as all casing method. But this method leaves casing in ground after the casting of concrete. If the rotation torque of the driving machine is large it can excavate to soft rock. It has advantage when the sub-soil strata has very hard intermediate layers. Permissibility of sub-soil strata is not required for required. Applying the thick steel casing, it can excavate very deep depth. It can be applied to the length more than 60m as well. This category of piles have steel casing left in the ground from pile top to bottom. However, it is considered as RCC pile due to its small friction between steel casing and casted concrete.

vi) Methodologies and structural details shall be referred to IRC:78-2014 and IRC:122-2011.

6) Precast piles

i) Pre-fabricated piles are divided into four categories, such as driven pile, inter-excavation piles, pre-boring piles, and steel piles filed with soil cement. Construction procedure differs by each category as follows.

ii) Driven piles are installed the piles without excavating the sub-soil. Instead of excavating the sub-soil, this method drives the piles by hammer of by vibration. It can be applied to many types of sub-soil strata without large size gravel, or very hard intermediate layer. It is very quick to drive and usually economical. However, the sound and vibration to surrounding area is very large and being avoided to apply in urban area especially the method by hammer. The materials are whether steel, RCC, or PSC.

iii) Inner excavation piles are installed by hammer or vibration after excavating the soil within the cross section of the pile by excavating drills. After installing the pre-fabricated piles at the indicated height on the bearing layer, the bottom end of the pile shall be firmly set by impact of hammer, injecting cement milk, or injecting cement concrete. The materials are whether steel, RCC, or PSC.

iv) Pre-boring piles are installed into the cement soil after excavating the shaft slightly bigger than the prefabricated shaft. Unification of prefabricated pile and cement soil surrounded/filled bear the load of the bridge. The materials are whether steel or RCC, however, there are steel material, such as H-beam is used for main body member of the pile.

v) Steel pile filled with soil cement are combined pile with soil-cement and steel piles. This steel pile has ribs on the surface as shear connector to secure the shear force transmitted between two materials. It is considered as a type of steel piles because the main body member is steel pile.

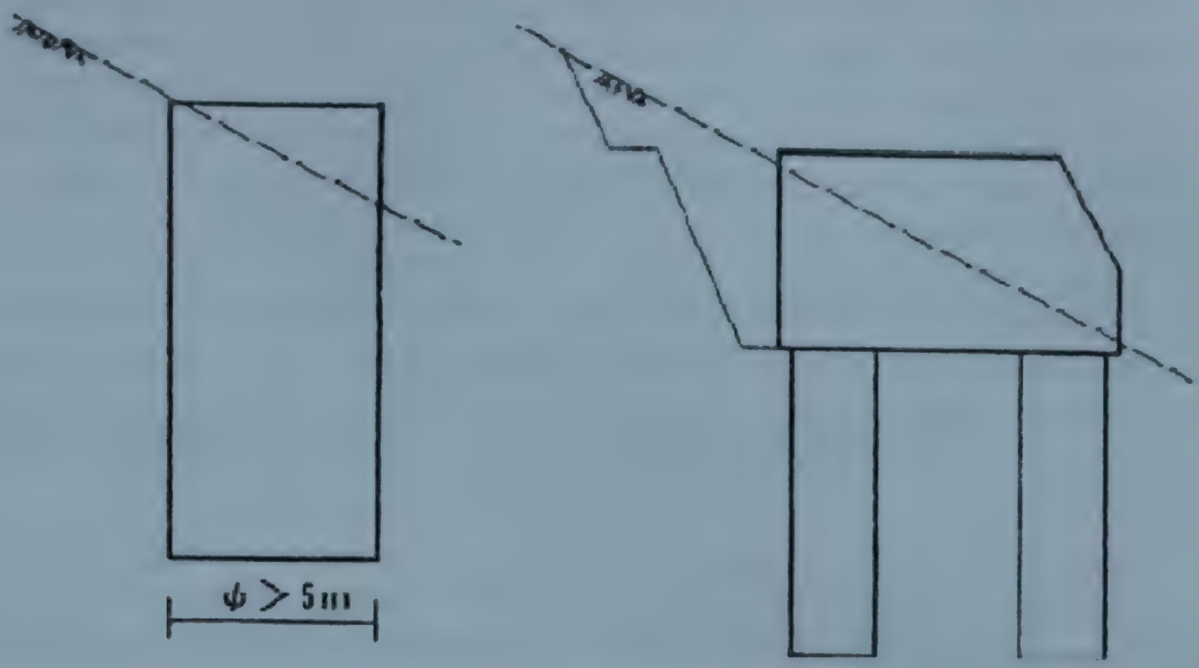
vi) Methodologies and structural details shall be referred to IRC:78-2014. If it is RCC structure, IRC:122-2011 shall also be referred to. If it is steel structure, IRC:24-2010 shall also be referred to.

(4) Deep Foundation

1) Deep foundation is basically applied for the foundation of the substructure of the bridge located on the slope steeper than 10 degrees where excavating machine/equipment is difficult to access or placed. This foundation is one of the methods of cast-in-situ pile foundation mainly excavated by man power or small exactor.

2) Pile diameter is larger than 2.0m and the length is basically up to 10 times long of diameter.

3) If the diameter is 5m or more, it is basically divided into two types, one is single cylindrical deep foundation and another is multiple combined deep foundation.



single cylindrical deep foundation multiple combined deep foundation

(Source: Design Specification of Civil Works, Kyushu Regional Development Bureau, MLIT, Government of Japan)

Figure 3-16: Typical Profile of Deep Foundation

4) Deep foundation is applied where the underground water level is basically lower than bearing layer. It means that it if very difficult to apply at the location near the river flow which increases water level considerably during heavy rain. However to protect the pile wall from water leakage and collapse, requires temporary retaining wall by the size of diameter. Diameter 2.5m and smaller requires liner plate, Diameter smaller than 5.0m requires mortar lining or shotcrete, 5.0m and larger requires shotcrete and rock bolts.

5) There in not any IRC Code, specification, manual, and guideline for designing deep foundation. However, IRC:112-2011 shall be referred to as RCC structure. And also there are descriptions in Specifications for Highway Bridges of Japan (Part IV Substructures), Chapter 15 'Design of Deep Foundation', as follows,

Quoted from Specifications for Highway Bridges of Japan (Part IV Substructures)

CHAPTER 15 Design of Deep Foundations

15.1 Basic Requirements of Design

(1) In verification of deep foundations under ordinary conditions, under storm conditions, or under Level I earthquake conditions, the following requirements shall be satisfied:
   1) The vertical unit subgrade reaction on the deep foundation bottom shall be equal to or smaller than the allowable vertical unit bearing capacity of the foundation bottom ground specified in Clause 15.4.1.
   2) The shear subgrade reaction on the deep foundation bottom shall be equal to or smaller than the allowable shear resistance force of the foundation bottom ground specified in Clause 15.4.2.
   3) The displacement of the deep foundations shall be equal to or smaller than the allowable displacement specified in Clause 9.2.
   4) The unit stress occurring in each member of the deep foundations shall be equal to or smaller than the allowable unit stress specified in Chapter 4.

(2) Verification of deep foundations under Level 2 earthquake conditions shall be performed in accordance with provisions specified in Clause 15.8.

This chapter is targeted for specific deep foundations. They will be constructed on a slope with a ground-surface inclination angle of 10 degree or larger by the deep foundation method, and their depth of embedment shall be generally larger than their foundation width. In the deep foundation method, drilling and earth retaining are performed in parallel in relatively robust ground with a low groundwater level; the condition of the bearing layer is directly checked by visual inspection and other approaches, and reinforced concrete is constructed under the same construction conditions and construction management as those exposed to air. In ground with low self-standing characteristics or with spring water, an earth retaining structure is formed with earth retaining material such as liner plates, and is formed with mortar lining or shotcrete in ground with high self-standing characteristics. Between these two cases, the shear ground resistance of the foundation peripheral surface and other properties are dealt with in different ways. Accordingly, an appropriate earth retaining structure must be selected in the design, with consideration given to ground conditions around the foundations.

Deep foundations are classified into two types: cylindrical deep foundations and combined deep foundations. Cylindrical deep foundations are constructed as a unit cylindrical structure, as is the case with caisson foundations and cast-in-site diaphragm wall foundations; combined deep foundations are constructed as a combined-pile structure consisting of multiple deep foundation piles rigidly connected to a footing. This chapter defines provisions on both types of deep foundations.

In some design concepts, temporarily attached members, such as lock bolts for earth retaining in the deep foundation method, are considered to be part of the resistance elements of foundations. However, there are various unclear points regarding temporarily attached members, such as construction quality, durability, and changes to their resistance mechanisms and reliability of their resistance mechanisms after a Level 2 earthquake. Accordingly, temporarily attached members are not included in the scope of this chapter. When their use is to be considered, it is necessary to examine the performance and other characteristics of each member separately.

(1) The provision specifies verification of deep foundations under ordinary conditions, under storm conditions, or under Level I earthquake conditions. The verification items have been defined with consideration given to the fact that deep foundations have the same resistance characteristics as those of other cylindrical-structure foundations, such as caisson foundations. In contrast, when combined deep foundations are to be used, the footing main body and the connections between the piles and the footing must be considered in the design, in addition to the verification items for cylindrical deep foundations.

    Since deep foundations are constructed on a slope, the effects of the slope must be appropriately taken into consideration in the design. Concrete methods for dealing with the effects of slopes are described in Clause 15.2 and the following sections.

(2) Verification of deep foundations under Level 2 earthquake conditions shall be performed in accordance with the basic approaches specified in Chapters 6, 12, and 13 of Part V: Seismic Design. Concrete calculation models, methods for determining parameter values, the definition of the yield of foundations, and the like are specified in Clause 15.8 of this volume.

Figure C.15.1.1 gives a standard design calculation flowchart for deep foundations for piers. In the design of deep foundations, the foundations shall not be permitted to develop primary plasticity for the reason described in Clause 15.8.1.

As is the case with an example of a pre-boring pile shown in (1) 1) of Clause 9 .1, when it is expected that the foundations will develop plasticity under Level 2 earthquake conditions, leading to distinct change in resistance characteristics, it is desirable to verify performance under ordinary conditions, under storm conditions, and under Level I earthquake conditions; performance should also be verified on the assumption of a case where hard rock po1iions will develop plasticity when subject to the effects of a Level 2 earthquake, leading to strength degradation.

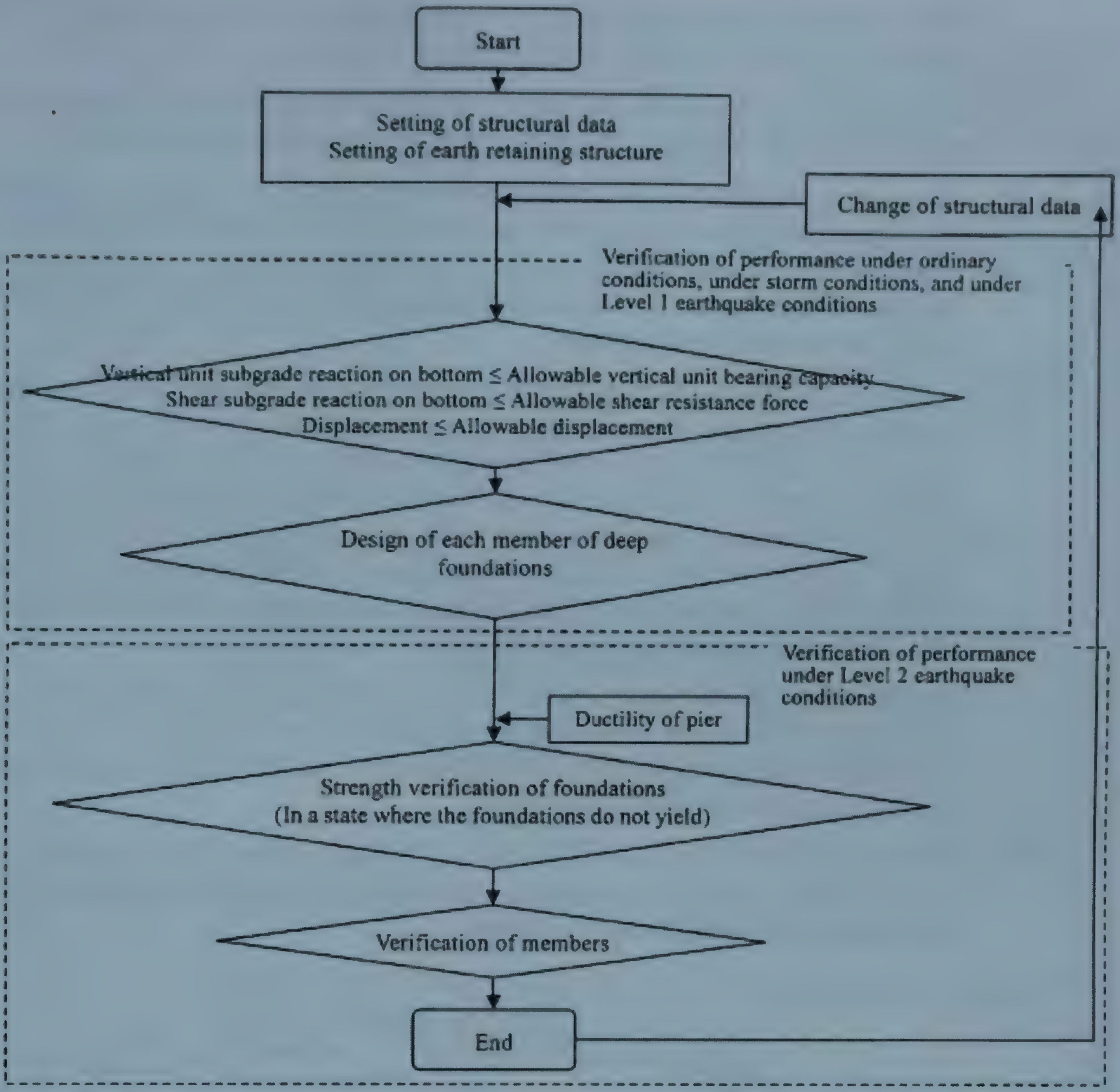

Start
Setting of structural data
Setting of earth retaining structure
Change of structural data
Verification of performance under ordinary conditions, under storm conditions, and under Level 1 earthquake conditions
Vertical unit subgrade reaction on bottom ≤ Allowable vertical unit bearing capacity
Shear subgrade reaction on bottom ≤ Allowable shear resistance force
Displacement ≤ Allowable displacement
Design of each member of deep foundations
Verification of performance under Level 2 earthquake conditions
Ductility of pier
Strength verification of foundations
(In a state where the foundations do not yield)
Verification of members
End

Figure C.15.1.1 Design Calculation Flowchart for Deep Foundations for Piers

15.2 Load Distribution

(1) The vertical subgrade reaction on the foundation bottom ground shall be used to resist vertical loads in principle.

(2) The vertical subgrade reaction and shear subgrade reaction on the foundation bottom ground and the horizontal subgrade reaction on the front ground shall be used to resist horizontal loads in principle.

(3) When an earth retaining structure expected to contribute to the shear resistance of the foundation peripheral-surface ground is to be used in natural ground with high self-standing characteristics, the horizontal shear subgrade reaction on the foundation side ground and the vertical shear subgrade reaction on the foundation peripheral-surface ground can be taken into consideration in addition to the resistance considered in (1) and (2).

The stability of deep foundations shall be evaluated with consideration given to contributions from ground resistance elements on the foundation bottom and peripheral surface, in the same manner as for caisson foundations. There are six types of ground resistance elements, as shown in Figure C.15.2.1.

Table-C.15.2.1 shows how to model the flexural rigidity of the foundations and the ground resistance elements to perform safety calculation. For deep foundations, the vertical ground resistance of the foundation bottom and the horizontal ground resistance of the foundation front shall be determined with consideration given to the effects of the slope.

(1) and (2) The provisions specify how to distribute loads in an earth retaining structure formed with earth retaining material, such as liner plates.

With regard to resistance to vertical loads, only the vertical subgrade reaction on the foundation bottom ground is considered, as is the case with caisson foundations.

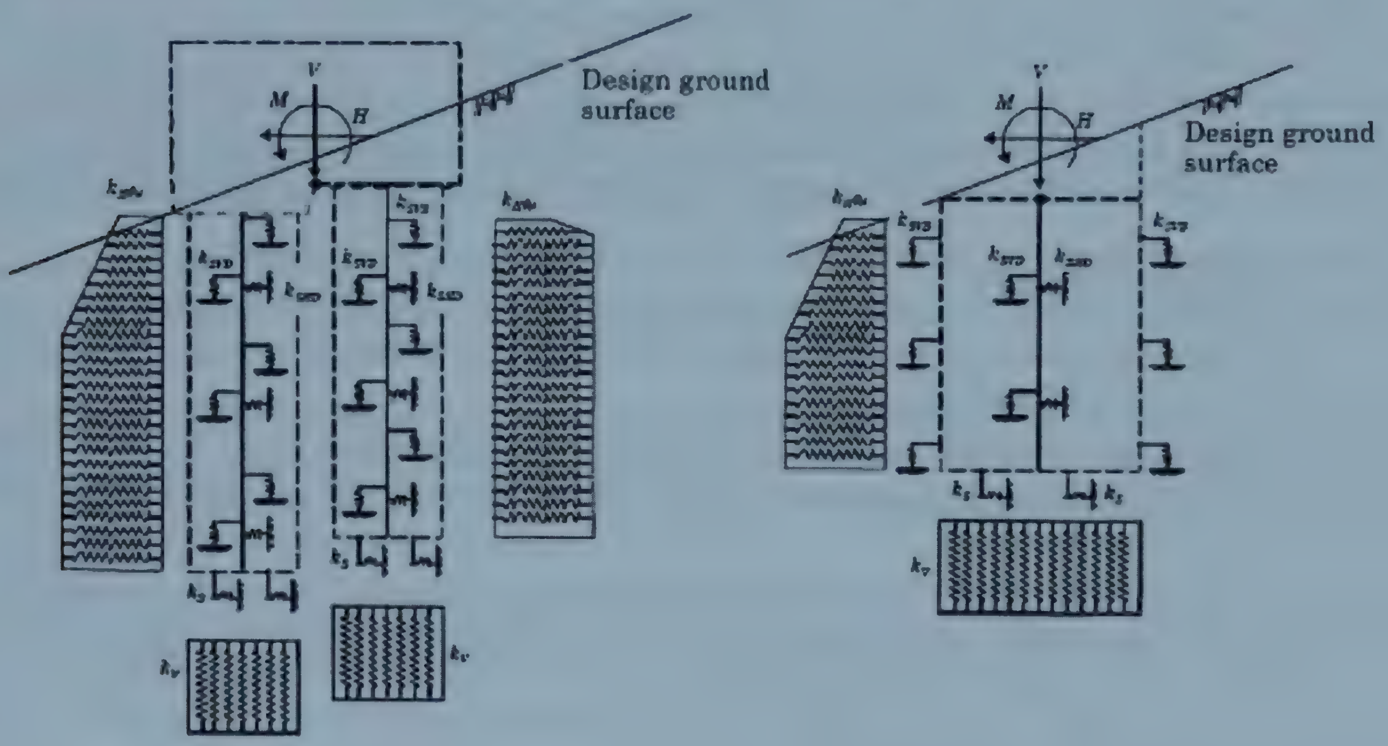
With regard to resistance to horizontal loads, the horizontal subgrade reaction on the foundation front ground shall be considered in the portion on or below the design ground surface.

(3) Mortar lining and shotcrete enhance the firmness of ground, contributing to integration between natural ground and foundations. The provision specifies that when an earth retaining structure formed with such a material is to be used for natural ground with high self-standing characteristics, the shear subgrade reactions on the foundation side ground and peripheral-surface ground can be assumed on the basis of verification by previous experiments.

In such earth retaining, drilling is performed within the range of depth where the stability of natural ground can be ensured. Subsequently, mortar lining or shotcrete with strength equal to or higher than that of the foundation body is used to form an earth retaining structure as a torus. The torus has a thickness equal to or larger than a specific value and is closely attached to the natural ground. This structure contributes to generation of the shear sub grade reaction on the foundation peripheral-surface ground, as shown in Figure C.15.2.1; it has been demonstrated by previous experiments that shear subgrade reaction equivalent to that generated in cast-in-place piles and cast-in-site diaphragm wall foundations can be assumed in this structure.

When an earth retaining structure is to be used, on the assumption of the shear sub grade reaction on the foundation peripheral-surface ground, evaluation of the self-standing characteristics of foundation peripheral-surface ground plays a markedly important role. The method for and considerations regarding such evaluation are given in Clause 15.10.

When earth retaining material, such as liner plates, is to be used in parallel with mortar lining or shotcrete under some stratal conditions, shear ground resistance can be partially considered for some arrangements of the earth retaining structure.



(a) Load Distribution in Combined Deep Foundations

(b) Load Distribution in Cylindrical Deep Foundations

$k_{H\Omega_y}$: Coefficient of horizontal subgrade reaction on foundation front

k_S: Coefficient of shear subgrade reaction on foundation bottom

k_{SVB}: Coefficient of vertical shear subgrade reaction on foundation front and back

k_{SVD}: Coefficient of vertical shear subgrade reaction on foundation side

Figure C.15.2.1 Ground Resistance Elements

Table C.15.2.1 Stability Calculation Model

		Verification of performance under ordinary conditions, under storm conditions, and under Level 1 earthquake conditions	Verification of performance under Level 2 earthquake conditions
Stiffness of foundations		- Linear	- The degradation of flexural rigidity is considered.
Ground resistance element	Vertical ground resistance of foundation bottom	- Linear	- Bilinear type - The upper limit is determined by the maximum vertical bearing capacity of the foundation bottom.
	Horizontal shear ground resistance of foundation bottom	- Linear	- Bilinear type - The upper limit is determined by the shear resistance force of the foundation bottom.
	Horizontal ground resistance of foundation front	- Bilinear type * - The upper limit is determined by the intensity of passive earth pressure on the foundation front ground subject to the effects of the slope.	- The same as on the left
	Horizontal shear ground resistance of foundation side **	- Bilinear type * - The upper limit is determined by the maximum skin friction force.	- The same as on the left
	Vertical shear ground resistance of foundation front and back **	- Bilinear type * - The upper limit is determined by the maximum skin friction force.	- The same as on the left
	Vertical shear ground resistance of foundation side **	- Bilinear type * - The upper limit is determined by the maximum skin friction force.	- The same as on the left

* The model for dealing with the behavior of hard rock shall be capable of evaluating the peak shear strength of the rock and the effects of strength degradation that occurs after the peak strength is reached.

** The resistance can be assumed when an earth retaining structure that allows shear resistance of ground around foundations to be assumed, such as mortar lining and shotcrete, is to be used.

15.3 Geometries and Arrangement of Deep Foundations

(1) The geometries of deep foundations shall be determined with consideration given to construction conditions in addition to the shapes and dimensions of piers or abutments on the foundations, the stability of the foundations, the effects of the slope, and unit stress occurring in each portion of the foundations.

(2) Arrangement in combined deep foundations shall be determined with consideration given the dimensions and quantity of deep foundation piles, the effects of the slope, construction conditions, and other factors; this should be done in such a manner that the deep foundation piles will receive loads uniformly to the extent possible under long-term persistent loads, with no excessive concentration on specific piles.

(1) The size of a foundation body must be determined with consideration given not only to safety sufficient for the stability of the foundations but also to the geometries of the bridge body supported by the foundations. Generally, the geometries of deep foundations are determined with consideration given to the following factors:

1) Foundation diameter

Cylindrical deep foundations must be sufficiently large for the stiffness of the bridge body and must have dimensions that allow the longitudinal reinforcements of substructure bodies to be firmly anchored. Accordingly, their foundation, diameter shall be 5 m or larger, with consideration given also to past application records. When combined deep foundations are used, checking on drilling conditions, bearing layer conditions and formation of the foundation body will be performed inside the borehole. Accordingly, their foundation diameter has been set to 2 m or larger in actual applications to ensure safety and workability.

As shown in Figure C.15.3.1, the foundation diameter inside the earth retaining structure shall be used as the design diameter for stability and section calculation. However, when liner plates are to be used, the diameter with respect to the axis line of the plates can be used only in stability calculation.

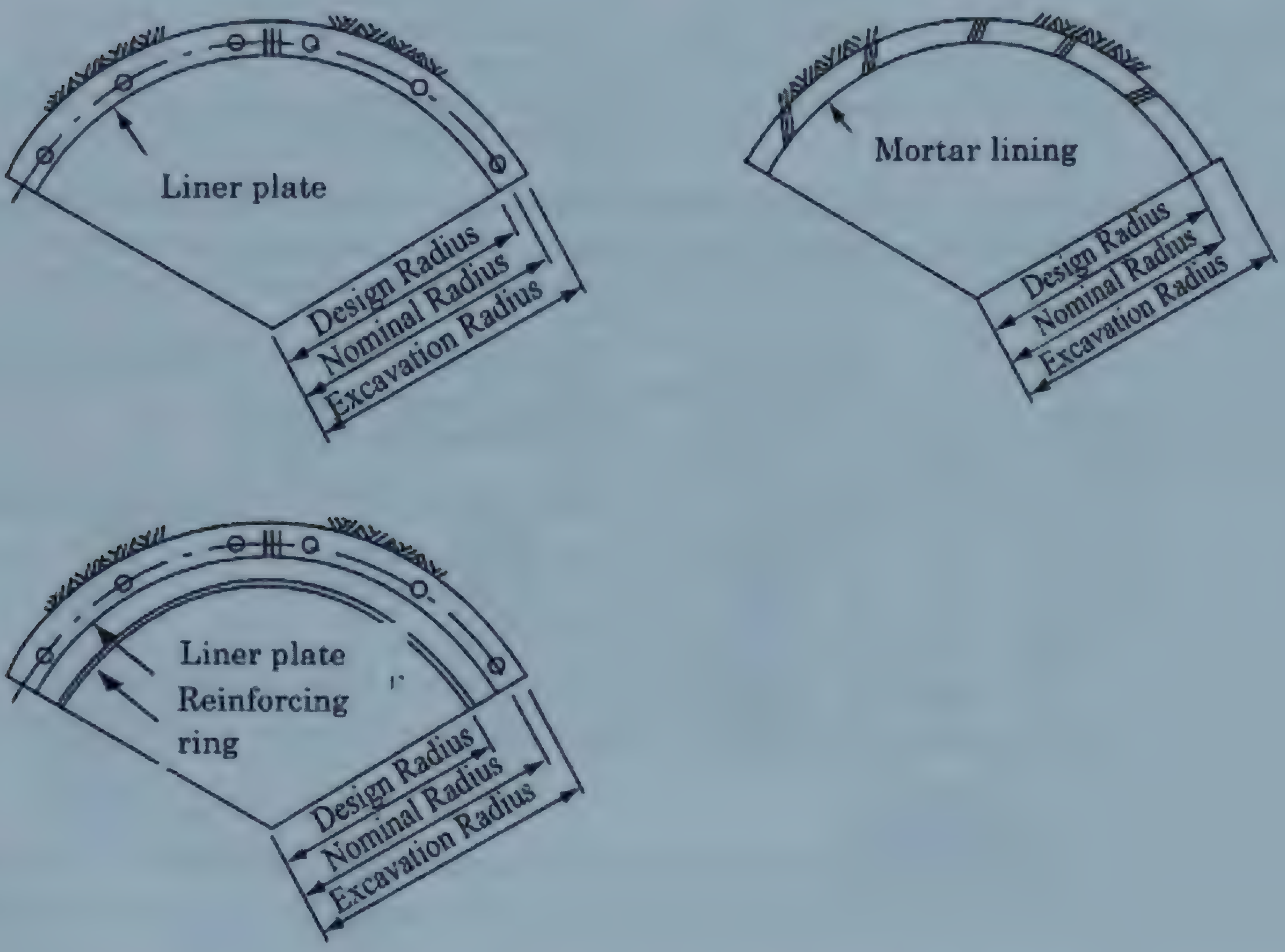


Figure C.15.3.1 Design Diameter of Deep Foundation Pile Diameter

2) Sectional form

Foundation bodies generally have circular cross sections, which are structurally advantageous in earth retaining used for drilling. When cross sections other than circular ones are to be used for some reason, such as construction restraints, the stability and other performance of earth retaining in the drilling must be examined separately in each case.

3) Effective depth of foundation embedment

It is desirable to set the effective depth of foundation embedment to a value roughly equal to or larger than the foundation diameter so as to satisfy premises for stability calculation, such as the flexural rigidity of the foundation body and ground resistance. However, since deep foundations are installed on slopes and require work in their boreholes, the depth of their embedment must be determined with consideration given to construction safety. According to construction records, it is up to approximately 30 m.

(2) The provision states how to arrange deep foundation piles in combined deep foundations. Deep foundations must be arranged with consideration given to various conditions such as the geometries of substructure bodies, the dimensions and quantity of piles, the effects of adjacent deep foundation piles, the effects of the slope,-and construction conditions; this should be done in such a manner that each deep foundation pile will receive loads effectively under long-term persistent loads.

Since deep foundations are to be constructed on slopes, their performance, for example ground resistance, is relatively difficult to evaluate compared with that of foundations constructed on flatlands. According to cases reported on abutments with single-row deep foundation piles, foundation front slopes became unstable due to natural phenomena such as earthquakes and rainfall, leading to disasters. Therefore, when combined deep foundations are to be used for an abutment on a slope, it is desirable to design four or more combined-pile structures, each consisting of multiple deep foundation piles, in both the longitudinal and transverse directions of the bridge. This is to ensure effective complementarity and substitutability if the surrounding ground has become unstable.

Generally, it is desirable to set the minimum spacing between deep foundation piles to a distance approximately two times that of the pile diameter as a measure against effects brought about when natural ground becomes less firm due to construction of adjacent foundations. These effects include degradation of ground resistance and destabilization of the slope on which deep foundation piles are installed.

The distance between the deep foundation piles and the footing edge shall be determined in accordance with provisions specified in Clause 12.3.

15.4 Allowable Bearing Capacity of Ground

15.4.1 Allowable Vertical Unit Bearing Capacity of Foundation Bottom Ground

(1) Allowable vertical unit bearing capacity of foundation bottom ground

The allowable vertical unit bearing capacity of foundation bottom ground shall be determined with consideration given to the ultimate unit bearing capacity of the foundation bottom ground and the settlement of foundations. In this process, the allowable vertical unit bearing capacity shall be determined in such a manner that the safety factors shown in Table-15.4.1 will be appropriately ensured for the ultimate unit bearing capacity of the foundation bottom ground.

Table-15.4.1 Safety Factor

Under ordinary conditions	Under storm conditions or under Level 1 earthquake conditions
3	2

(2) Ultimate unit bearing capacity of foundation bottom ground

1) The ultimate unit bearing capacity of foundation bottom ground shall be determined on the basis of the results of appropriately performed ground surveys, such as the form and dimensions of foundations, the depth of foundation embedment, and the effects of the slope.
2) When the ultimate unit bearing capacity of foundation bottom ground is determined with consideration given to the effects of the slope and on the basis of the ultimate bearing capacity of the foundation bottom ground obtained in accordance with the provisions in (2) of Clause 11 .4.1, then 1) shall be regarded as satisfied.

(1) As is the case with caisson foundations, it is specified that the allowable vertical unit bearing capacity should be determined with consideration given to the ultimate unit bearing capacity of foundation bottom ground and the settlement of foundations. To prevent excessive settlement of foundations, it is desirable to suppress the allowable vertical unit bearing capacity under ordinary conditions in the same manner as

for the upper limit to pneumatic caisson foundations shown in Figure C.11.4.1. The value of 1.5 times that of this unit bearing capacity can be used as the upper limit of the vertical unit bearing capacity under storm conditions and under Level 1 earthquake conditions. Also in this case, the allowable vertical unit bearing capacity of the foundation bottom ground must be determined in such a manner that the required safety factors will be ensured for the ultimate unit bearing capacity stated in (2).

When ground is bedrock, its ultimate bearing capacity varies depending on its physical conditions, cracks and rifts, and evaluation of soil constants is affected by various elements of uncertainty. Accordingly, it is difficult to estimate the ultimate bearing capacity using a bearing capacity estimation formula. Therefore, as is the case with spread foundations, it is desirable to suppress the maximum unit subgrade reaction roughly to upper limits corresponding to the uniaxial compressive strength of the base rock in Table-C.15.4.1. Compared with spread foundations, deep foundations have a large effective depth ratio (the ratio of the effective embedment depth of the foundations to the foundation diameter) and are not easily affected by physical conditions such as cracks and rifts occurring when foundation bottom ground becomes less firm. Accordingly, the values shown in Table-C.15.4.1 have been determined on the basis of the results of plate loading tests on hard and soft rock at depths of or 5 m or more.

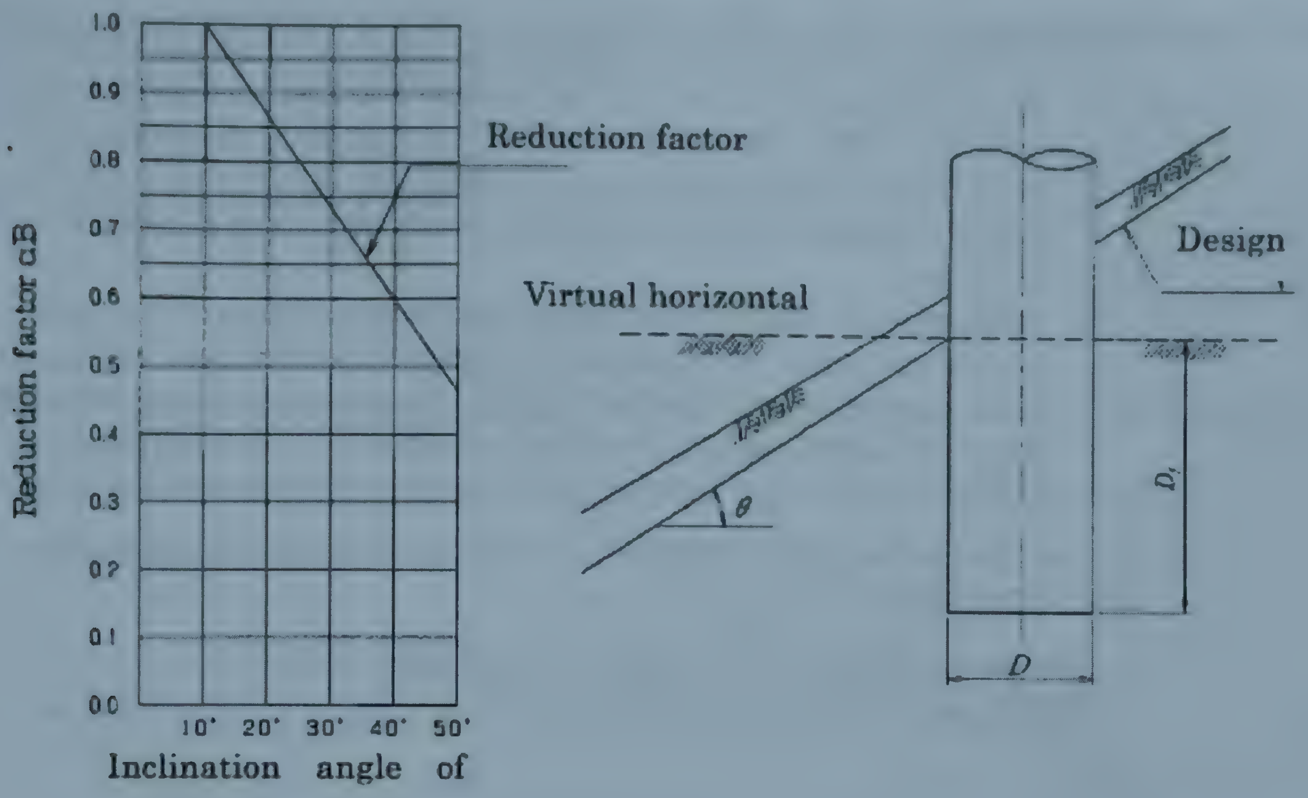
The upper limit of the .maximum unit subgrade reaction thus specified is sufficiently small compared with the allowable unit bearing capacity determined with consideration given to the effects of the slope and on the basis of the ultimate bearing capacity of the foundation bottom ground obtained in accordance with the provisions in (2) of Clause 11.4.1. Therefore, the effects of the slope need not be taken into consideration.

Table-C.15.4.1 Upper Limits of Maximum Unit Subgrade Reaction on Bedrock

Type of bedrock	Maximum unit subgrade reaction on foundation bottom ground (kN/m^2)		Reference uniaxial compressive strength (MN/m^2)
	Under ordinary conditions	Under storm conditions or under Level 1 earthquake conditions	
Soft rock	2,000	3,000	1 to 10
Hard rock	2,500	3,750	10 or higher

(2) 2) The ultimate unit bearing capacity of foundation bottom ground shall be determined as a value that is reduced with consideration given to the effects of the slope from the ultimate bearing capacity of horizontal ground obtained by Equation (11.4.1) for caisson foundations.

Equation (11.4.1) is applied on the premise of horizontal ground extending over a sufficient area. When it is to be applied to foundations on a slope, since foundation front ground has only a finite area, the value obtained by Equation (11.4.1) must be reduced. Figure C.15.4.1 (a) shows typical values of the reduction factor (the ratio of the ultimate unit bearing capacity of foundations on a slope to its ultimate unit bearing capacity on horizontal ground) estimated on the premise that deep foundations with a foundation diameter of approximately 2 to 15 m and an effective embedment depth equal to or larger than the foundation diameter are constructed on a slope available as a bearing layer with an inclination angle of the ground surface of 50 or smaller Under the above-mentioned conditions, the ultimate unit bearing capacity can be determined by multiplying the reduction factor aB given in Figure C.15.4.1 (a). Reduction factor aB shall be determined from inclination angle 8 of the design ground surface in the foundation front shown in Figure C.15.4.1 (b). However, under conditions beyond the above-mentioned range, it is necessary to examine each case separately to determine the reduction factor. Effective embedment depth Df of foundations in Equation (11 .4.1) shall be the distance from the foundation bottom to the horizontal ground (virtual horizontal ground) lying at the intersection between the design ground surface and the foundation front, as shown in Figure C.15 .4.1 (b). Since these are deep foundations, as is the case with the caisson foundations, the effects of eccentric inclination of loads and the scale effect of the bearing capacity factor can be ignored.



(a) Reduction Factor α_B

(b) Definition of Inclination Angle θ of Design Ground Surface

Figure C.15.4.1 Reduction Factor α_B of Ultimate Unit Bearing Capacity of Foundation Bottom Ground

15.4.2 Allowable Shear Resistance Force of Foundation Bottom Ground

(1) Allowable shear resistance force on foundation bottom ground shall be determined in such a manner that the safety factor shown in Table-15.4.2 will be ensured for the shear resistance force acting between the foundation bottom and the ground.

Table-15.4.2 Safety Factor

Under ordinary conditions	Under storm conditions or under Level 1 earthquake conditions
1.5	1.2

(2) Shear resistance force on foundation bottom ground shall be determined with sufficient consideration given to ground conditions. When shear resistance force on foundation bottom ground is determined by Equation (11.4.2), this requirement shall be deemed to be satisfied.

(1) and (2) These provisions have been specified on the basis of provisions on caisson foundations.

In calculation of shear resistance force on foundation bottom ground, values shown in Table-C.10.3.4 can be used as the frictional force and adhesive force between the foundation bottom and the ground. The "effective loading area of a foundation bottom" refers to the area of the base of the deep foundations, excluding loose parts.

15.5 Coefficients of Subgrade Reaction and Upper Limits of Unit Subgrade Reaction

15.5.1 Coefficients of Subgrade Reaction

(1) The coefficients of subgrade reaction to be used for the design of deep foundations shall be determined appropriately for load distribution specified in Clause 15 .2 on the basis of sufficient examinations of the results of ground surveys and soil tests, and with consideration given to conditions such as the effects of the slope.

(2) When the coefficients of subgrade reaction are determined in the following ways, (1) shall be deemed to be satisfied:

1) The coefficients of subgrade reaction to be used for the design of deep foundations shall be the coefficients of vertical subgrade reaction and horizontal shear subgrade reaction on the foundation bottom and the coefficient of horizontal subgrade reaction on the foundation front; the coefficients shall be determined on the basis of sufficient examinations of the results of ground surveys and soil tests, and with consideration given to conditions such as the effects of the slope.
2) When an earth retaining structure specified in (3) of Clause 15.2 is to be used, the coefficient of horizontal shear subgrade reaction on the foundation side and the coefficient of vertical shear subgrade reaction on the foundation peripheral surface in addition to the coefficients of subgrade reaction stated in 1) shall be detem1ined.

(2) The coefficients of subgrade reaction to be used for the design of deep foundations can be determined in the following ways:

① Coefficient of vertical sub grade reaction on foundation bottom
The coefficient of vertical sub grade reaction on the foundation bottom shall be determined by Equation (C.9.6.1).

② Coefficient of horizontal shear sub grade reaction on foundation bottom
The coefficient of horizontal shear subgrade reaction on the foundation bottom shall be determined by Equation (C.11.5.1).

③ Coefficient of horizontal sub grade reaction on foundation front
The coefficient of horizontal subgrade reaction on the foundation front shall be determined by Equation (C.15.5.1).

$$k_{H\theta\mu} = \alpha_{H\theta}\mu k_H \cdots\cdots (C.\ 15.5.1)$$

where

$K_{H\theta\mu}$: Coefficient of horizontal subgrade reaction determined with consideration given to the effects of the slope and adjacent piles;
$a_{H\theta}$: Correction factor for taking the effects of the slope into consideration for coefficient of horizontal subgrade reaction;
μ: Correction factor for taking the effects of adjacent piles into consideration for coefficient of horizontal subgrade reaction;
k_H: Coefficient of horizontal subgrade reaction determined by Equation (C.11.5.2); in this calculation, *ak* shall be set to 1.5.

④ Coefficient of horizontal shear subgrade reaction on foundation side
The coefficient of horizontal shear subgrade reaction on the foundation side shall be determined by Equation (C.11.5.4).

⑤ Coefficient of vertical shear subgrade reaction on foundation front and back
The coefficient of vertical shear subgrade reaction on the foundation front and back shall be determined by Equation (C.11.5.7).

⑥ Coefficient of vertical shear subgrade reaction on foundation side
The coefficient of vertical shear sub grade reaction on the foundation side shall be determined by Equation (C.11.5.8).

For cylindrical deep foundations with a circular or oval foundation cross section, stability can be estimated from effective loaded width *Be* and effective side face width De calculated by Equation (C.11.5.9), as is the case with caisson foundations. However, the estimation shall be performed for the actual form of the foundation bottom, and the flexural rigidity of foundations shall be also calculated for the actual form of the foundation bottom.

Since foundation front ground on slopes has only a limited area, the coefficients of subgrade reaction on the foundations are expected to be smaller than those oh foundations on horizontal ground. Accordingly, the coefficients of horizontal subgrade reaction shall be reduced by multiplying correction factor aH8 calculated by Equation (C.15.5.2), as shown in Equation (C.15.5.1).

$$\left.\begin{array}{ll} \alpha_{H\theta}=0. & (0 \leqq \alpha_H < 0.5) \\ \alpha_{H\theta}=0.3\log_{10}\alpha_H+0.7 & (0.5 \leqq \alpha_H < 10) \\ \alpha_{H\theta}=1.0 & (10 \leqq \alpha_H) \end{array}\right\} \cdots\cdots (\text{C. } 15.5.2)$$

where

- $a_{H\theta}$: Correction factor for talcing the effects of the slope into consideration for coefficient of horizontal subgrade reaction;
- a_H: Ratio between length of horizontal soil cover to the slope and foundation diameter D.

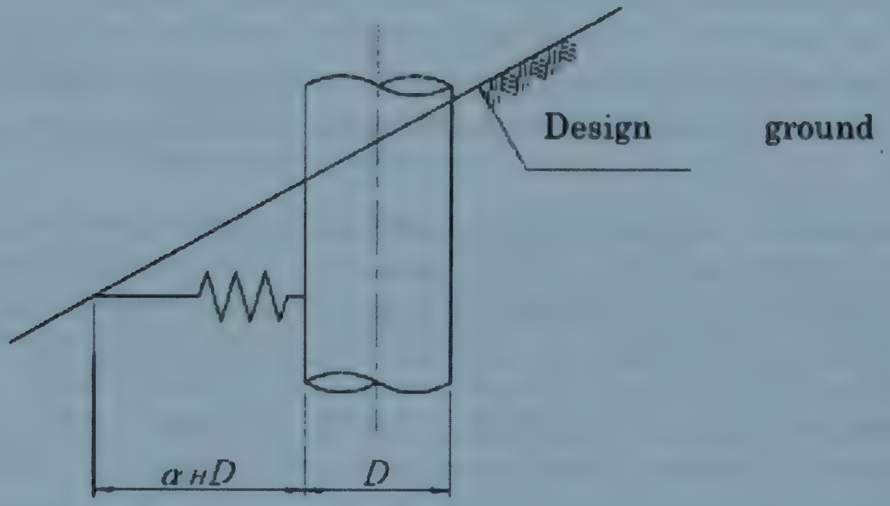


Figure C.15.5.1 Ratio UH between Length of Horizontal Soil Cover to the Slope and Foundation Diameter D

As is the case with pile foundations, when the pile center distance of deep foundations is small, the effect of the pile group must be taken into consideration. However, since deep fow1dations are to be constructed on slopes, the magnitude of the reduction in response to the effects of adjacent piles must be strictly evaluated compared with that required for pile foundations on horizontal ground. Accordingly, when deep foundations are to be arranged in multiple rows with a pile center distance shorter than five times that of the foundation diameter, the coefficients of horizontal subgrade reaction will be reduced by multiplying correction factor μ calculated by Equation (C.15.5.3), as seen in Equation (C.15.5.1).

$$\mu=\frac{1}{6}\sqrt{\left(\frac{P_1}{D}+1\right)\left(\frac{P_2}{D}+1\right)} \cdots\cdots (\text{C. } 15.5.3)$$

where

- μ: Correction factor for taking the effects of adjacent piles into consideration for coefficient of horizontal subgrade reaction;
- P_1: Center distance of piles adjacent in the direction perpendicular to loading (m);
- P_2: Center distance of piles adjacent in the direction of loading (m);
- D: Foundation diameter (m)

15.5.2 Upper Limit of Horizontal Unit Subgrade Reaction on Foundation Front

(1) Toe upper limits of horizontal unit subgrade reaction on the foundation front under ordinary conditions, under storm conditions, or under Level 1 earthquake conditions shall be the value obtained by dividing the passive soil pressure intensity of foundation front ground by a correction factor shown in Table-15.5.1.

Table-15.5.1 Correction Factor

	Under ordinary conditions	Under storm conditions or under Level 1 earthquake conditions
Horizontal unit subgrade reaction on foundation front	3	2

(2) The passive soil pressure intensity of foundation front ground shall be appropriately determined on the basis of the results of appropriately performed ground surveys, and with consideration given to the inclination of the slope in the foundation front, the spread angle of the slip surface, and the effects of shear strength degradation occurring after the ground develops plasticity.

(1) Horizontal unit subgrade reaction on the foundation front shall be determined by design using a bilinear-type ground resistance model for evaluating partial plasticity development in the ground, so that the displacement will be suppressed to a level equal to or smaller than the allowable displacement that does not lead to phenomena such as increase in residual displacement.

(2) Deep foundations are constructed on slopes and often require embedment into bedrock. Accordingly, passive soil pressure intensity must be determined with consideration given to the effects of the slope in the foundation front, the three-dimensional spread of the slip surface, and shear strength degradation due to plasticity development in the ground.

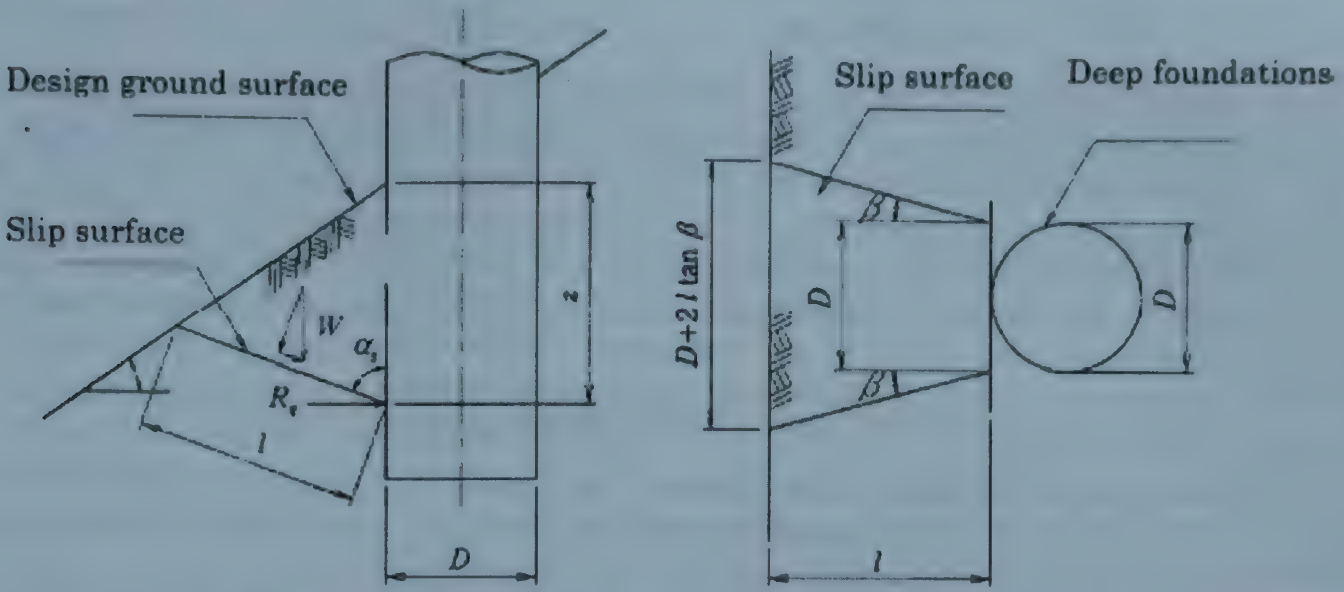
As seen in Equation (C.15.5.4), the passive soil pressure intensity of foundation front ground is determined by depth-direction differentiation of the ultimate horizontal bearing capacity obtained from soil mass slip. The ultimate horizontal bearing capacity at a specific depth can be determined by Equation (C.15.5.5) as the minimum value of the shear resistance of the flat slip surface shown in Figure C.15.5.2. The concept of the slip surface used in Equation (C.15.5.5) and shown in Figure C.15.5.2 has been defined on the basis of the results of previous horizontal loading tests on deep foundations.

$$p_p = \frac{\partial R_q}{\partial z}\frac{1}{D} \quad \cdots\cdots \text{(C. 15.5.4)}$$

$$R_q = \frac{W\left(\cos\alpha_S + \sin\alpha_S \tan\phi\right) + cA}{\sin\alpha_S - \cos\alpha_S \tan\phi} \quad \cdots\cdots \text{(C. 15.5.5)}$$

where

P_p: Passive soil pressure intensity at depth z (kN/m^2);
R_q: Ultimate horizontal bearing capacity obtained from slip of soil mass from ground surface to depth z (kN); it shall be obtained by Equation (C.15.5.5);
z: Depth of target slip surface (m);
D: Foundation diameter (m);
W: Weight of soil above slip surface at depth z (kN); it shall be obtained from angle of slip surface as in Figure C. 15.5.2;
A: Area of slip surface at depth z (m^2); it shall be obtained from angle of slip surface as in Figure C. 15.5.2;
φ: Shear resistance angle of ground (0);
c: Cohesion of soil (kN/m^2);
a_S: Angle of slip surface; it shall be the angle at which Rq reaches its minimum value



β: Spread angle of the slip surface (°); generally, it can be set as follows:
Earth and sand, and soft rock: $\beta = 30 + \phi/3$;
Hard rock: $\beta = \phi/3$

Figure C.15.5.2 Slipping Soil Mass

When foundation peripheral-surface ground is bedrock, residual strength tends to decrease markedly after the peak strength has been reached. Accordingly, for a foundation peripheral surface under such ground conditions, different passive soil pressure intensity must be used depending on whether the foundation front ground has developed plasticity. Specifically, peak strength can be used as the shear strength of the ground when the foundation front ground has not developed plasticity, and residual strength (φ_{res}, C_{res}) can be used when the foundation front ground has developed plasticity. In this process, it is desirable to set residual strength Cres of cohesion to O from a safety perspective. However, when strength after plasticity development has been directly confirmed by a test such as the multi-stage triaxial compression test described in Reference Material 1, it can be set to a value appropriately within the range of $0 \leqq Cres \leqq c/3$. Residual strength φ_{res} of shear resistance angle can be set to $(2/3)\varphi$>.

Generally, "robust bedrock for which residual strength is used as passive soil pressure intensity obtained after foundation front ground has developed plasticity" refers to bedrock consisting of hard rock with a uniaxial compressive strength of 10 MN/m^2 or higher, as shown in Table-C.15.4.1. However, for bedrock that markedly decreases in residual strength in spite of its uniaxial compressive strength being equal to or lower than the reference values for hard rock, it is desirable to use residual strength as passive soil pressure intensity obtained after the bedrock has developed plasticity.

When adjacent foundations are present or when stratal structure changes on the slip surface, the upper limit must be determined with consideration given to the effects of these conditions.

15.5.3 Upper Limits of Unit Subgrade Reaction on Foundation Side and Foundation Peripheral Surface

When an earth retaining structure specified in (3) of Clause 15.2 is to be used, the upper limits of the horizontal unit shear subgrade reaction on the foundation side and the vertical unit shear subgrade reaction on the foundation peripheral surface under ordinary conditions, under storm conditions, or under Level 1 earthquake conditions shall be determined by dividing the maximum unit skin friction at each location by a correction factor shown in Table-15.5.2

Table-15.5.2 Correction Factor

		Under ordinary conditions	Under storm conditions or under Level 1 earthquake conditions
Horizontal unit shear subgrade reaction on foundation side		1.5	1.1
Vertical unit shear subgrade reaction on foundation peripheral surface	Pushing reaction	3.0	2.0
	Pulling reaction	6.0	4.0

As described in (3) of Clause 15.2, when an earth retaining structure such as mortar lining is used and can be expected to contribute to the shear resistance of foundation-peripheral surface ground, unit shear subgrade reaction on the foundation side and the foundation peripheral surface can be assumed. As is the case with the horizontal unit subgrade reaction on the foundation front described in Clause 15.5.2, the unit shear subgrade reaction on the foundation peripheral surface shall be expressed as a bilinear-type model, and its upper limit shall be determined by dividing the maximum unit skin friction by each correction factor shown in Table-C.15 .5 .2.

The maximum unit skin friction shall be obtained by Equation (C.15.5.6).

The upper limits for sandy soil and cohesive soil shown in Equation (C.15.5.6) have been defined on the basis of Equation (C.14.5 .1) so as to apply to highly self-standing, dense sandy soil and hard cohesive soil.

The upper limits for soft rock and hard rock have been defined with consideration given to the margin of safety on the assumption that the unit shear stress when the foundations start slipping is approximately one-third of the uniaxial compressive strength. However, since hard rock decreases in strength after its peak strength, the upper limit for hard rock has been determined on the assumption that its residual strength after plasticity development will be one-tenth of the peak strength, as is the case with the passive soil pressure intensity of foundation front ground.

Soft rock and hard rock are classified depending on the reference uniaxial compressive strength shown in Table-C.15.4.1.

$$
\left.
\begin{array}{l}
\text{Sandy soil} \\
\qquad f = \min\,[5N,\ (c + p_0 \tan\phi)] \leq 200 \\
\text{Cohesive soil} \\
\qquad f = c + p_0 \tan\phi \leq 150 \\
\text{Soft rock} \\
\qquad f = c + p_0 \tan\phi \leq 300 \\
\text{Hard rock} \\
\qquad \text{Elastic region: } f = c + p_0 \tan\phi \leq 1{,}500 \\
\qquad \text{Plastic region: } f_0 = c_{res} + p_0 \tan\phi_{res} \leq 150 \\
\qquad\qquad\qquad (0 \leq c_{res} \leq c/3 \text{ and } \phi_{res} = 2\phi/3)
\end{array}
\right\} \cdots\cdots (\text{C}.15.5.6)
$$

where

f. Maximum unit skin friction (kN/m^2);
f_o: Maximum unit skin friction in plastic region of hard rock (kN/m^2);
N: N value determined by standard penetration test;
c: Cohesion of soil and rock (kN/m^2);
p_0: Static soil pressure intensity acting on wall surface (kN/m^2);
φ. Shear resistance angle of soil and rock {°);
C_{res}: Residual strength of rock (kN/m^2);
φ_{res}= Residual shear resistance angle of rock (°)

15.6 Calculation of Sectional Force, Unit Subgrade Reaction, and Displacement

(1) The sectional forces, unit subgrade reaction, and displacement of deep foundations shall be calculated with appropriate consideration given to the characteristics of the foundations and ground.

(2) When the calculation is performed in the following manner, (1) shall be deemed to be satisfied:

The sectional forces, unit subgrade reaction, and displacement of deep foundations shall be calculated on the assumption that the foundations are an elastic body and with the aid of an analytical model that is capable of evaluating the non-linearity of ground. In this process, cylindrical deep foundations shall be regarded as a finite-length beam, and combined deep foundations shall be regarded as a rigid-frame structure. In the calculation, horizontal unit subgrade reaction on the foundation front, horizontal unit shear subgrade reaction on the foundation side, and vertical unit shear subgrade reaction on the foundation peripheral surface shall not exceed the corresponding upper limits specified in Clause 15.5.

(1) and (2) Safety calculation for cylindrical deep foundations shall be performed with consideration given to the effects of the slope described in Clause 15.4, in the same manner as for caisson foundations. Safety calculation for combined deep foundations shall be performed with the aid of a rigid-frame model of deep foundation piles connected via a footing; before being connected to the footing, the piles shall be modeled in the same manner as for cylindrical deep foundations.

If looseness or pulling occurs on the foundation bottom due to rotation of foundations, vertical subgrade reaction and shear subgrade reaction cannot be assumed in the loosened portions.

Since combined deep foundations are constructed on slopes, they are often structurally different from pile foundations constructed on horizontal ground. Depending on the inclination of their design ground surface and bearing layer, the length of deep foundation piles are often changed, and footing used for combined deep foundations often has differences in level. In such cases, since different deep foundation piles bear different loads, calculation must be performed with the aid of a model that is capable of appropriately deali.t,1g with differences in load distribution. For example, design can be performed on the basis of appropriate evaluation of loads and sectional forces by model analysis, such as analysis using a space rigid-frame model and analysis using plane rigid-frame models. In the analysis using plane rigid-frame models, the proportion of loads distributed to each pile row arranged in the direction perpendicular to loading is calculated separately with the aid of a plane rigid-frame model; and then, analysis of each pile row is performed with the aid of a plane rigid-frame model within the plane.

When the proportion of loads distributed to some piles becomes small due to some physical properties of the foundations, such as the relative position of piles with respect to the slope and differences in pile length, design without any consideration will result in markedly imbalanced foundations. However, to ensure the complementarity and substitutability required when the surrounding ground becomes unstable, design must be performed in such a manner that deep foundation piles will receive loads uniformly to the extent possible, with no excessive load concentration on specific piles. Accordingly, it is desirable to take measures such as setting the lower limit of the load home by each pile row to the average load, which is obtained by dividing the total loads acting on the foundations by the number of the pile rows.

15.7 Verification of Each Member of Foundations

15. 7.1 Combined Deep Foundations

(1) It shall be verified that deep foundation piles are safe against the combination of loads acting after their installation.

(2) It is desirable to design footings for combined deep foundations as rigid-frame structures that are rigidly connected to deep foundation piles.

(3) The connections between deep foundation piles and footings shall be formed by rigid connection in principle, and it shall be verified that the connections are safe against forces occurring in the connections.

(1) Design shall be performed with consideration given to the axial forces, bending moments, and shear forces that are calculated in accordance with provisions specified in Clause 15.6.

(2) Footings for combined deep foundations are structurally different from pile foundations constructed on horizontal ground and often have complex shapes as seen in multilevel footings. In addition, the rigid-body assumption may not be satisfied in the relative relationship with the stiffness of deep foundation piles. Accordingly, it is necessary to use models suitable for structural conditions for actual structures to design foundations, for example, as a rigid-frame beam member consisting of deep foundation piles and a footing, not as a cantilever specified in Clause 8.7.3 and Clause 8.7.4. For deep foundation piles receiving loads distributed at different proportions as described in Clause 15.6, design must be performed with the aid of an analytic model that is capable of evaluating sectional forces appropriately, such as a space rigid-frame model. When a footing is to be designed to have differences in level, it is desirable to take structural measures such as putting haunches and reinforcing bars in place, and to make considerations in design calculations so that stresses will not concentrate locally in section-area changing portions, thus leading to damage.

(3) It shall be verified that the connections between deep foundation piles and footings in caisson type combined pipe foundations are safe against the pushing forces, pulling forces, and horizontal and bending moments that act on the heads of the deep foundation piles. Specifically, safety can be verified by examination of vertical and horizontal punching shear on deep foundation piles at footing ends.

In some cases, a model such as a rigid-frame structure model, suitable for structural conditions for the actual structure, is used in the design. As described in (2), this approach to dealing with structural conditions is different from that for typical footings for pile foundations. In those cases, the verification necessary for the structural conditions shall be performed.

When anchoring the longitudinal reinforcements of deep foundation piles to a footing, since the anchorages are the connections of the rigid-frame structure as described in (2), it is desirable to ensure the anchorage length shown in Figure C.7.6.3 (b) from the footing bottom surface.

15.7.2 Cylindrical Deep Foundations

(1) It shall be verified that the general portion of the foundation body is safe against the combination of loads acting after their construction.

(2) The connections of the cylindrical deep foundations to the bridge body shall have a structure that can reliably transmit loads from the bridge body to the foundation body.

(1) As is the case with Clause 15.7.1, design shall be performed with consideration given to the axial forces, bending moments, and shear forces calculated in accordance with provisions specified in Clause 15.6.

(2) Figure C.15.7.1 outlines the structure of a connection of the cylindrical deep foundations to the bridge body. The connections of the cylindrical deep foundations to the bridge body are portions that have a function equivalent to that of footings or pile caps that transmit loads from superstructures and substructures. However, the connections are different from footings and pile caps in that the foundation body lies below the connections, in contact with their bottoms. Differently from footings and other members, design is not performed on their cross sections. Accordingly, it is necessary to consider their thicknesses and reinforcement arrangements separately for each case so that loads from the bridge body can be reliably transmitted to the foundation body.

As seen in Equation (C.15.7.1), the thickness of bridge-body connections shall be determined in such a manner that the required anchorage length will be ensured for both the longitudinal reinforcements of the bridge body and the longitudinal reinforcements of the foundation body. The anchorage length of the longitudinal reinforcements of the bridge body in the bridge-body connections shall be obtained as the sum of the anchorage length of the reinforcements and one-half of the bridge-body length in its short side direction.

$$t = \mathrm{Max}\,[L_1,\ L_2] \quad \cdots\cdots \quad (C.15.7.1)$$

where

t: Thickness of bridge-body connection (mm);

L_1: Thickness necessary for ensuring anchorage length of longitudinal reinforcements of bridge body in bridge-body connection (mm); it shall be obtained as the sum of the thickness sufficient for ensuring the anchorage length of reinforcements determined by Equation (7.8.1) and one-half of the bridge-body length in its short side direction;

L_2: Thickness necessary for ensuring anchorage length of longitudinal reinforcements of foundation body in bridge-body connection (mm); it shall be the anchorage length of reinforcements determined by Equation (7.8.1).

With consideration given to concrete compaction at placing joints, it is desirable to use semicircular hooks at the ends of the longitudinal reinforcements of the bridge body. However, when semicircular hooks cannot be used as for S0490, it is desirable to give consideration to concrete compaction, attaching rectangular hooks in an arrangement without overlapping.

As shown in Figure C.15.7.1, the anchorages of the longitudinal reinforcements of the bridge body shall be arranged in a stagger pattern so that anchorages will not concentrate on a specific cross section. It is desirable to set the vertical spacing at ends to approximately 1 m or larger, as is the case with lap splices stated in Clause 7 .8 or cutoffs whose positions overlap with those of construction joints.

As is the case with footings and other members, bridge-body connections are designed separately as unit members, and no reinforcements are arranged in them. Accordingly, it is desirable to arrange reinforcing bars on the connection tops and within the connections as shown in Figure C.15. 7 .1 so that loads from the bridge body can be reliably transmitted to the foundation body. ·

Use of reinforcing bars can be determined with past cases of reinforcement arrangement in similar structures as a guide, and through analysis that is capable of examining damage to connections, such as FEM analysis; consideration is to be given to the state of stresses transmitted from the longitudinal reinforcements of the bridge body to the connections, and the effectiveness of the reinforcements to be arranged in the deep foundations.

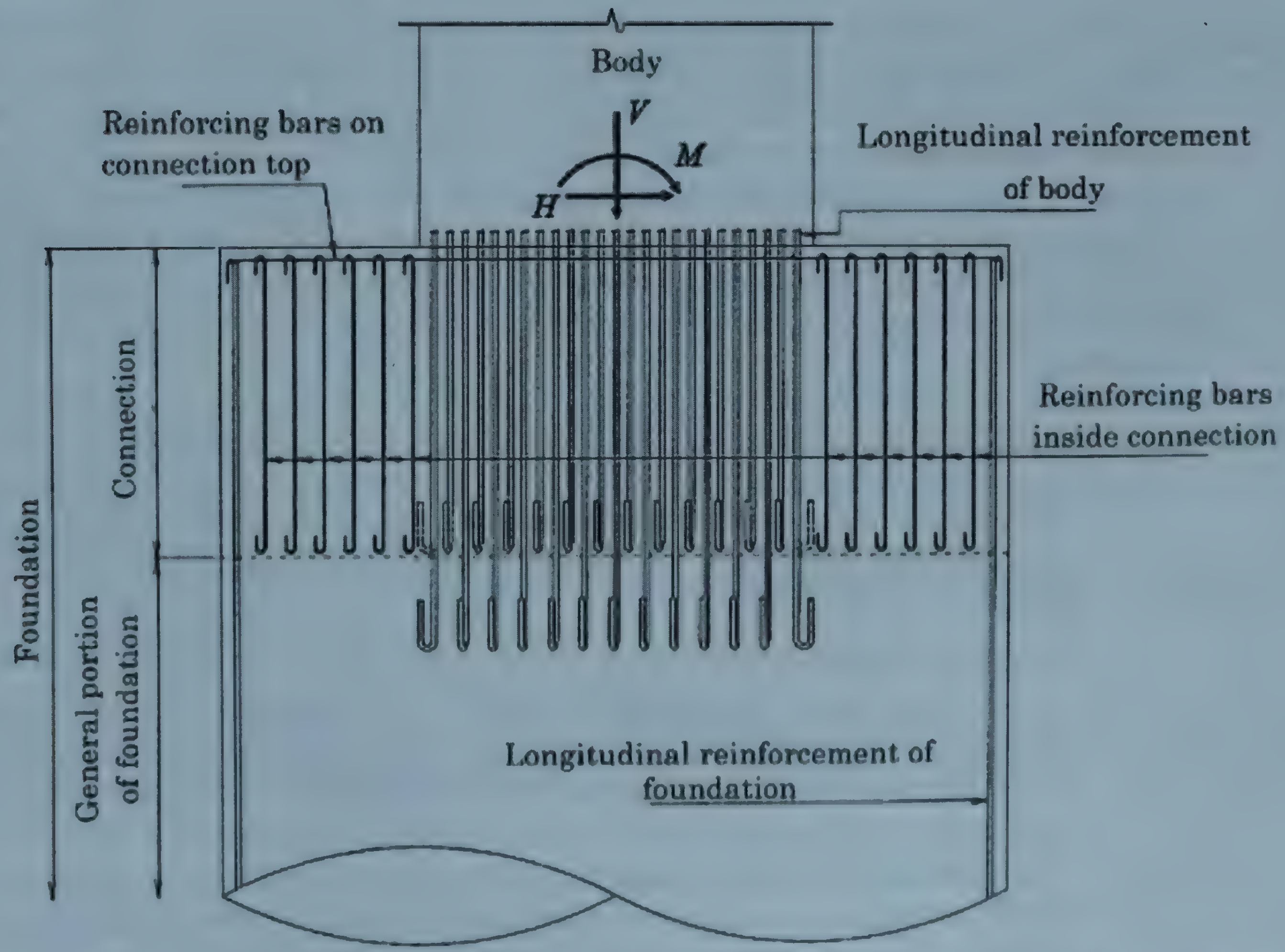

Body
V
M
H
Reinforcing bars on connection top
Longitudinal reinforcement of body
Reinforcing bars inside connection
Foundation
Connection
General portion of foundation
Longitudinal reinforcement of foundation

Figure C.15.7.1 Example of Structure of Each Portion of Cylindrical Deep Foundations

15.8 Verification of Performance under Level 2 Earthquake Conditions

15.8.1 Bases of Verification

(1) The sectional forces, unit subgrade reaction, and displacement that occur in deep foundations for piers when loads specified in (2) of Clause 6.4.7 of Part V: Seismic Design act on the foundations shall be calculated in accordance with provisions specified in Clause 15.8.3, and it shall be verified that the deep foundations will not reach the yield specified in Clause 15.8.2.

(2) The strength of deep foundations against sectional forces occurring in each member shall be verified in accordance with provisions specified in Clause 15.8.4.

How to verify the performance of deep foundations under Level 2 earthquake conditions by the ductility design method is specified in Chapters 6, 12, and 13 of Part V: Seismic Design. The provisions of this section specify calculation of the sectional forces, unit subgrade reaction, and displacement that occur in deep foundations, and the yield of the foundations. For cylindrical deep foundations, their performance can be verified in the same manner as for caisson foundations. For combined deep foundations, deep foundation piles shall be modeled in the same manner as for cylindrical deep foundations; foundation performance shall then be verified in the same manner as for caisson foundations, through calculation using a rigid-frame model of piles connected via a footing.

There are various factors of uncertainty regarding the behavior of deep foundations after plasticity development: for example, shear strength degradation after plasticity development in rock and destabilization of slopes under earthquake conditions. Accordingly, design must be done in such a manner that foundations will not reach the yield specified in Clause 15.8.2.

15.8.2 Yield of Foundations

The yield of deep foundations shall be defined as a state where horizontal displacement at the acting points of the inertia forces of superstructures starts to increase rapidly due to plasticity development in the foundations, plasticity development in the ground, or looseness of the foundations.

"The yield of foundations" refers to a state where horizontal displacement at the acting points of the inertia forces of superstructures starts to increase rapidly in the load-displacement relationship in the entire behavior of the foundations. Possible factors leading to rapid increases in the displacement of deep foundations are plasticity development in the foundations, plasticity development in foundation front ground, and looseness of foundation bottom. However, since deep foundations yield in markedly different patterns under different conditions, such as the resistance characteristics of the surrounding ground, it is difficult to specify a common definition of their yield. Therefore, the yield shall be determined by a yield judgment method such as the log P-log S method, on the basis of calculation of the load-displacement relationship in the overall behavior of the foundations.

15.8.3 Calculation of Sectional Forces, Unit Subgrade Reaction, and Displacement

(1) The sectional forces, unit subgrade reaction, and displacement of deep foundations shall be calculated with appropriate consideration given to the characteristics of the foundations and ground.

(2) When the calculation is performed in the following manner, (1) shall be deemed to be satisfied:

The sectional forces, unit subgrade reaction, and displacement of deep foundations shall be calculated, with the aid of an analytical model that is capable of evaluating the non-linearity of the foundations and ground; in this process, cylindrical deep foundations shall be regarded as a finite-length beam, and combined deep foundations shall be regarded as a rigid-frame structure. In the calculation, vertical unit subgrade reaction and unit shear subgrade reaction on the foundation bottom, horizontal unit subgrade reaction on the foundation front, horizontal unit shear subgrade reaction on the foundation side, and vertical unit shear subgrade reaction on the foundation peripheral surface shall not exceed their corresponding upper limits.

(2) The strength and displacement of deep foundations shall be calculated basically by the method specified in Clause 15.6.

For the calculation, the characteristics of foundation members and the resistance of ground surrounding the foundations can be specified in the following ways:

1) Characteristics of foundation members

The characteristics of foundation members shall be determined in the same manner as for caisson foundations stated in Clause 11.8.4.

2) Resistance of ground surrounding foundations

In verification by the ductility design method, each ground resistance element shall be dealt with as a bilinear-type element, in accordance with provisions specified in Clause 15.6. The upper limits of unit subgrade reaction shall be set in the following ways i) to iv):

i) Vertical unit subgrade reaction on foundation bottom
For sandy soil, sand gravel, or cohesive soil, the upper limit of vertical unit subgrade reaction on the foundation bottom shall be set to the ultimate unit bearing capacity of foundation bottom ground specified in Clause 15.4.1. For bedrock, it shall be set to the value of three times that of the upper limit of the maximum unit subgrade reaction under ordinary conditions shown in Table-C.15.4.1.

ii) Unit shear subgrade reaction on foundation bottom
The upper limit of unit shear subgrade reaction on the foundation bottom shall be set to the passive soil pressure intensity obtained by Equation (C.11.8.11).

iii) Horizontal .unit subgrade reaction on foundation front
The upper limit of horizontal unit subgrade reaction on the foundation front shall be obtained by Equation (C.15.5.4).

iv) Horizontal unit shear subgrade reaction on foundation side and vertical unit shear subgrade reaction on foundation peripheral surface
The upper limits of horizontal unit shear subgrade reaction on the foundation side and vertical unit shear subgrade reaction on the foundation peripheral surface shall be set to the maximum unit skin friction obtained by Equation (C.15.5.6).

15.8.4 Verification of Members

It shall be verified that sectional forces calculated in accordance with provisions specified in Clause 15.8.3 as those occurring in each member of deep foundations do not exceed the strength of the member.

1) Verification of performance of foundation body against bending moments
Since the yield of foundations is determined with consideration given to the bending moments of the foundation body, as is the case with caisson foundations, this verification can be omitted.

2) Verification of performance of foundation body against shear forces
It shall be verified that shear forces occurring in the foundation body do not exceed the shear strength of the foundation body specified in Clause 5.2.3.

3) Verification of footing of combined deep foundations
Sectional forces occurring in the footing shall be calculated in accordance with provisions specified in Clause 15.8.3, and verification of the footing shall be performed in accordance with provisions specified in Clause 8. 7.

4) Verification of connections between combined deep foundations and footing
Verification of the connections shall be performed in the same manner as Clause· 15. 7 .1.

15.9 Structural Details

(1) The cover of reinforcements shall be determined with consideration given to conditions such as the earth retaining structure, the unevenness of natural ground, the configuration of the reinforcements, and durability. This requirement shall be regarded as satisfied when the cover of reinforcements is set to 70 mm or larger from the outer boundary of the design diameter.

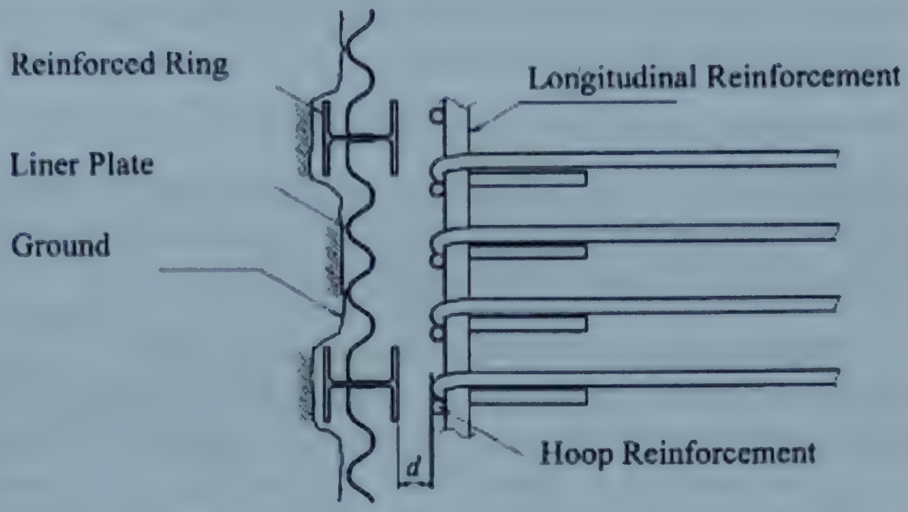


Figure 15.9.1 Minimum Cover d of Reinforcements

(2) Longitudinal reinforcements and shear reinforcing bars shall be arranged in the following ways, with consideration given to workability, so that they will function effectively:

1) Longitudinal reinforcements shall be arranged in accordance with the provisions of (3) 1) of Clause 12.11.2.
2) Longitudinal reinforcements shall be jointed with mechanical joints in principle.
3) For cylindrical deep foundations, hoop reinforcements of one-fourth or more the amount of the longitudinal reinforcements shall be arranged across the entire length of the foundations.
4) Hoop reinforcements for combined deep foundations shall be arranged in accordance with the provisions of (3) 2) of Clause 12.11.2.

Deep foundations, whose reinforcements are assembled in their boreholes, are significantly different from piers in the reinforcement assembly process. For piers, reinforcements can be assembled from both of the inner and outer sides of their cross sections, whereas for deep foundations, the outermost hoop reinforcements are assembled first, and then longitudinal reinforcements are built up. Accordingly, for reinforcement arrangement in deep foundations, it is necessary to make sufficient considerations not only for the arrangement of reinforcements in their completed form, but also for the structure and arrangement necessary for reinforcement assembly during construction.

(1) The cover of deep foundations must be determined so as to satisfy the required value at the most demanding location of the reinforcements on the outermost boundary of the foundation body. When structural steel, such as a reinforcing ring, is to be used as an earth retaining material, it must be noted that the required cover needs to be ensured from the internal surface of reinforcing bars.

(2) 1) For workability, 22-mm or larger longitudinal reinforcements shall be used, and clearance shall be set to either two times that of the diameter of the reinforcements or two times that of the largest dimensions of the coarse aggregate, whichever is larger.

2) It is specified that mechanical joints should be used as joints for longitudinal reinforcements in deep foundations in principle, to ensure safety of reinforcement assembly work in boreholes.

3) The provision specifies the minimum amount of reinforcements of hoop reinforcements used in cylindrical deep foundations. Since it is desirable to ensure a certain level of deformability also for cylindrical deep foundations, the provision defines the minimum amount of reinforcements of hoop reinforcements.

15.10 Design of Earth Retaining Structure

(1) An earth retaining structure for deep foundations shall be selected appropriately for the ground conditions, so that construction can be performed safely and reliably.

(2) An earth retaining structure for deep foundations shall be designed in such a manner that it will be safe against loads acting during construction and so that performance of the bearing mechanism of the foundations can be reliably delivered after completion.

(1) When determining an earth retaining structure for deep foundations, it is essential to select a structure that can ensure safety during construction.

Earth retaining materials such as liner plates have been widely used up to now, and function as earth retaining structures even in highly disintegrable slope taluses or in ground with a certain level of spring water present. In contrast, since mortar lining and shotcrete are applied after drilling and function as earth retaining structures only after their strength has developed, they can be used only on the premises of natural ground that will be self-standing during this period and of the absence of spring water that will disturb construction. Since deep foundations are designed on the assumption of shear ground resistance of the foundation peripheral surface, if the earth retaining structure is switched to liner plates or the like during construction, the dimensional data of the foundations will be affected.

Accordingly, an earth retaining structure must be carefully selected on the basis not only of a sufficient understanding of the strength and self-standing properties of the natural ground and the conditions of ground water and spring water, but also on the basis of an understanding of the effects of assumed shear resistance on basic data.

When the target natural ground includes highly disintegrable ground, such as slope taluses, or contains spring water, mortar lining and shotcrete may not be applied across the entire length of the foundations. In such cases, it is possible to consider partially using liner plates in parallel with using mortar lining, shotcrete, or the like appropriately for the conditions of the target stratum.

(2) An earth retaining structure for deep foundations must be designed to ensure safety in the drilling work. In addition, it must be designed with consideration given to the effects of consumption quality on the bearing mechanisms of the foundations, such as the ground resistance of the foundation peripheral surface.

The following are design considerations for the safety of the drilling work:

When liner plates are to be used, design shall be performed to ensure safety against static soil pressure. Since liner plates have a thin-wall annular structure, they are basically designed with consideration given to buckling and compressive stresses. In addition, it must be noted that the inner surface formed by drilling is not a perfect circle, and it is necessary to consideration bending moments generated by the action of eccentric earth pressure due to geographical features. For a large effective foundation embedment depth or for a large foundation diameter, when safety cannot be ensured by use only of liner plates, reinforcing rings or the like must be used in parallel to ensure safety.

When mortar lining or shotcrete is to be used for deep foundation piles with a relatively small bore as seen in combined deep foundations, safety is often verified on the assumption of the solidification strength of mortar or shotcrete in accordance with a design method used for designing liner plates. When shotcrete is to be used for deep foundation piles with a large bore, it will not be independently used. In such cases, an earth retaining structure is often determined as a combination of shotcrete with Jock bolts and steel supports that are used for mountain tunnels, on the basis of a supporting pattern formulated by the NATM (New Austrian Tunneling Method).

When mortar lining or shotcrete is to be used to satisfy design premises such as reliable ground resistance after completion, it is necessary to specify conditions such as mix proportion so as to ensure strength equal to or higher than that of the foundation body.

Bibliography

1) Public Works Research Institute: Research on Evaluation of Unit Subgrade Reaction to Establish a Method for Vertical Safety Verification of Foundations on Bedrock, Technical Memorandum of PWRI No. 4222, March 2012

[Source: Specification for Highway Bridges of Japan (Part IV Substructures)]

(5) Caisson/well Foundation

1) General

i) Caisson foundation is applied basically to bear massive load transmitted to foundation from bridge. The cross section is usually large and it can be more than 50m of diameter and height. These massive size is applied for very long span bridge of which superstructure is often suspension bridge or cable styed bridge.

ii) It could be applied to mountain bridges as well when pile foundation is not appropriate to be applied, such as require numerous numbers of piles, and etc.

iii) There are two types of caisson foundation, one is pneumatic caisson and another is open caisson. The difference is how to handle underground water, Pneumatic caisson put pressure to the area near to excavation ground surface. Work is highly automated to protect workers from high air pressure. Open caisson only stop the underground water by retaining wall/water proofing sheets, etc. It lets the underground water leak from the wall of shaft up to certain quantity and pump it out to secure the safety and workability.

iv) Materials are RCC in most of the cases except massive caisson made by steel. However, these massive caisson foundation is applied seashore region. It is very rare to be applied for mountain bridges. Considering the fact mentioned above, it can be said that RCC is the material for caisson foundation of mountain bridges.

v) There are two IRC Codes to be referred to. One is IRC:78-2014 and another is IRC:112-2011. Methodologies and structural details are described in these two IRC Codes.

3.2.3.8 Approaches, River Protection, River Training Works, etc.

(1) General

1) Most of the mountain bridges cross over river flow and its substructure is facing the risk of scour and debris flow if it is located within the river flow.

2) If the river is primitive, river channel tends to shift by flood. It can shift for several 100m in some locations. Therefore, river training is necessary to protect the bridge from flood disaster. It leads to protect the lives and properties of people live around, natural environment of surrounding area, and maintain growth of regional economy.

(2) Approaches tends to come into river bank to shorten the bridge and reduce the construction cost. However, it can be easily washed away in flood with shift of river channel. Not only protection of approach road the needs of river training shall be studied and design the protection method if necessary.

(3) River Protection to cope with scour is highly necessary in most of the mountain bridges due to its high flood flow velocity. Not only the surrounding of substructure, Dikes, riverbed shall also be protected to protect the bridges.

(4) River Training Works is necessary when the river channel has risk to shift easily. Since flood velocity in mountain area in very high, most of the bridge facing risk of river channel shift. Required works to protect the bridge shall be planned and designed in this stage.

3.2.3.9 Software Application

(1) Detailed design shall be done by analysing the phenomenon that the bridges is facing by using appropriate software.

(2) Drawing and reports shall be digitized.

3.2.4 Detailed Cost Estimates

3.2.4.1 Cost Estimates and its Content

(1) In addition to bridges in plane or rolling terrain, the cost estimate shall be done for the specific contents for mountain bridges. However. It depends on the construction method which is selected in detailed design.

3.2.4.2 General Abstract of Cost

(1) Cost estimates shall be done carefully in construction planning for mountain bridges. It is as same as those bridge at coastal area, swam area, or crossing big river, which have severe natural condition.

3.2.4.3 Details of Cost of Each Major Head

(1) Some heads of cost estimates shall differ by the construction method and site condition of the mountain bridge. The responsible cost estimator and bridge engineer shall be familiar with the construction method and temporary works which is selected in detailed design.

3.2.4.4 Details of Quantities

(1) Quantity of some contents shall differ to the bridges in plain or rolling terrain due to its severe natural condition Necessary evidence to justify the increase of them shall be prepared.

3.2.4.5 Justification of Rates

(1) The unit costs of each material differs considerably by site and region in mountain bridge construction. Necessary evidence to justify the increase of them shall be prepared.

3.2.4.6 Foreign Exchange Component

(1) When the project has foreign exchange components, it shall be separately estimated. This is because to hedge exchange

3.2.5 Specification

3.2.5.1 All of the specification for the contractor to execute the construction shall be prepared in this stage.

3.2.6 Special Specifications

3.2.6.1 When the project has new or rare materials, equipment, facilities applied, special specification shall be prepared in this stage. This special applications shall be approved by implementation authorities.

3.2.7 Preparation of Detailed Drawings

3.2.7.1 General

(1) Drawings are not only the result of detailed design. It shall also show the site condition, bridges system. Details of each member, reliable evidence for calculating the project cost.

3.2.7.2 Drawings Sizes

(1) A3 size drawings are the most common in the construction sites these days. However, it shall be enlarged to A0 size to show wide area of the components shown it the drawings.

3.2.7.3 Component Parts of Bridge Project Drawings

(1) Components are describe in IRC SP:54-2018. However, the drawings to show the specific or abnormal condition of the site shall be added and prepared without a failure.

3.2.8 Detailed Project Report

3.2.8.1 General

(1) Detailed project report need to explain all the engineering, environmental, social, economic, and financial status and forecast of the project.

3.2.8.2 Contents of the Report

(1) Contents are shown on IRC SP:54-2018. However, the contents to show the specific or abnormal condition of the site shall be added and prepared without a failure.

3.2.8.3 Introduction to the Report

(1) Information which shall be given in this clause is shown on IRC SP:54-2018. However, the other important information to explain the outline of the project shall be added and prepared without a failure.

3.2.8.4 Site Selection

(1) Summary of the comparison studies shall be described chronologically to justify the site location of the bridge in this section. Selecting the site give large impact to project in mountain bridge projects. Therefore, it shall be prepared in details.

3.2.8.5 Topographical Survey

(1) Not only the topographic map which are prepared in the detailed design stage, new and important findings of the site shall be reported without failure. Methodologies and procedure of the survey and analysis of the survey result is also important to be described.

3.2.8.6 Hydraulic Data and Waterway Fixation

(1) Not only the design discharge data analysed by hydrological engineer, characteristics, new and important findings of the river flow shall be reported without failure. Methodologies and procedure of the survey and analysis of the survey result is also important to be described.

3.2.8.7 Design of Bridges Elements

(1) Summary of the comparison studies of bridge type and span arrangement shall be described chronologically to justify the site location of the bridge in this section. Bridge type and span arrangement give large impact to project in mountain bridge projects. Therefore, it shall be prepared in details.

(2) Each elements of the bridge design result, methodologies, procedure shall be describe in this clause. Spatial condition, material, construction method, etc. shall be clearly highlighted to show the difference to construction contractor.

3.2.8.8 Material and Resources

(1) List of the materials and resources with quantity shall be prepared. If there are some special materials or resources, it shall be clearly highlighted to show the difference to construction contractor.

3.2.8.9 Specifications

(1) Materials, equipment, facilities, tests, construction method, procurement shall be listed and if there does not exist standard specification, it shall be prepared for authority's approval.

3.2.8.10 Estimates

(1) Summary of cost estimate shall be demonstrated. Contents are shown on IRC SP:54-2018. However, the contents to show the specific results or preparation required for construction shall be added and prepared without a failure.

3.2.8.11 Construction programme/Methodology

(1) Construction method from accessing the site shall be prepared chronologically by steps. This will be evidence to justify the results of detailed design and also give previous advises to construction contractors to prepare the project. It can be revised or modify to improve the cost and efficiency.

3.2.8.12 Working Drawings

(1) Working drawing shall be submitted with the report for authority's approval. However, construction contractor shall prepare the working drawings by themselves right before the actual work after executing the site, material, procurement confirmation

(2) Not only working drawings but also drawings to explain the results shall be prepared.

3.2.8.13 Economic/Financial Analysis

(1) Results of the economic analysis shall be updated based on the results of detailed design to secure the economic feasibility of the project. Cost is not only the construction cost, environmental and social cost shall be included to analyse the social economic feasibility since the bridges are social property of the region/nation

3.2.8.14 Financial Forecast and Budget Provision

(1) Financial evaluation shall be reported with several assumption of operational organization and method of financial procurement. It is difficult to fix the detailed financial condition at this stage. Therefore, several alternatives shall be prepared to improve the reliability of the evaluation.

3.2.8.15 Environment Impact Assessment

(1) Mountain bridges tends to give severe impact to natural environment of the surrounding area, therefore results of environmental impact assessment shall be described in the report.

(2) Countermeasures and its results of applying it shall be explained in the report. Not only the methods but also the implementing schedule for the counter measures shall be described in the report.

3.3 Quality Assurance System

3.3.1.1 Bridge is integral system with various materials and members. Therefore, mal-quality of single member/material could be the trigger of entire system failure, which results in closure of the bridge for a long period. Controlling the quality of material to products both procured and fabricated is very critical for securing the quality of the entire bride.

3.3.1.2 Quality control system during the construction shall be prepared and regulation for construction contractor to prepare and obtain approval from authority before the commencement of each works/items in construction stage.

3.3.1.3 To secure the quality assurance system working properly, monitoring system shall be prepared for actual execution in the construction stage.

3.3.1.4 Additional control items and its specification shall be prepared to secure the quality at the site in hilly terrain, of which the construction method specially applied for mountain bridges, such as 'Bamboo Split Shaped Retaining Method', 'Weathering/Weatherproof Steel', and etc.

3.3.1.5 Control items related to slope shall be prepared for implementation of mountain bridges. Monitoring system of slope in surrounding area of the site shall be included to secure the site safety.

3.3.1.6 Natural environment shall controlled and monitored based on the EIA made for the project. It shall be more strict system is there is rare and endangered species. The responsible personnel of the project shall always keep in mind that there are higher opportunity of existence of rare and endangered species in the implementation of mountain bridges.

3.3.1.7 Necessity of safety control system shall be written in the quality assurance system. Actualization of risk of accident impacts negatively to the quality of construction works. It shall be noted that the change of natural environment are sudden in hilly area. More strict control is required at the site in hilly area.

3.4 Maintenance Manual

3.4.1.1 Maintenance manual for the entire bridge and also each member shall be prepared as a standard of the region, the entire route, or each operational organization.

3.4.1.2 Maintenance manual for the project shall be prepared for materials, members which are unique or not so common for bridges. Also if the bridge is facing regional environment which gives negative impact to the bridge, specific manual shall be prepared to mitigate the negative impact.

3.4.1.3 Mountain bridges tends to have longer spans and complicated structures, such as high pier, integrated structure of substructure and superstructure, statically indeterminate structures of high orders. These bridges are very difficult to be inspected in accessible distance, and every member requires longer period of durability. However, it shall be maintained to sustain the service level of the bridge equivalent to that of opening stage. Therefore requirements in maintenance shall be prepared as manual during the design stage by design engineer, and upgraded in construction stage by construction contractor/supervision consultant before its opening.

3.4.1.4 Selecting of the structure and material of the bridge is done in designing stage. Initial specification and durability shall be design to reduce the maintenance works.

3.4.1.5 During the construction stage, points to be carefully maintained in operation stage shall be added to the maintenance manual. There are often amendments of design, such as shape, materials, and specification of the members. These amendments shall be referred to in upgrading the maintenance manual.

3.4.1.6 Massive bridges with long span and high piers are very difficult to inspect and keep the sound condition for long period. Organization structure of maintenance, specifically for the emergency maintenance (inspection and temporary repair) is recommended to be implemented considering the structure and local environment of the bridge.

3.4.1.7 'GUIDELINE FOR OPERATION AND MAINTENANCE' which are prepared in The JICA Project is also available for reference.

3.5 Executive Summary

3.5.1.1 Executive summary shall be prepared for the report since the report has very large volume to report numerous contents which was studied and analysed in detailed design.

3.5.1.2 Executive summary shall not only explain the outline of the result of detailed design. It shall clearly highlight the specific or unique results, application, or findings in detailed design.

3.6 Seismic Design (added)

3.6.1 General

3.6.1.1 Mountain bridges in Himalayan Mountains and North-eastern Region faces the risk of massive earthquakes with complicated movement. Therefore special consideration shall be made in detailed design stage.

3.6.1.2 In this subchapter, the design policy and methodology of Japan which are described in 'Specification for Highway Bridges' are introduced.

3.6.1.3 Methodology of dynamic analysis is focused on this subchapter.

3.6.1.4 Important points to be considered for the complicated structures, such as Unsymmetrical Design/Span arrangement, High/very High piers, Integral structure of substructure and superstructure rigidly connected, are also explained.

3.6.1.5 Aseismic devices, such as Aseismic Bearing/Device and Bridge Fall Prevention Device are introduced. Aseismic Bearing/Device plays important role in reducing seismic force impacting the substructure and foundation and prevent fatal collapse of the bridge. Bridge Fall Prevention Device regulates the displacement of the superstructure not to fall down from the bridge seats of the substructures. These functions are 2 important functions to secure the function as the bridge for the regional road network even after the earthquake occurred. Securing the function as a bridge let the emergency/rescue vehicles to reach the disaster site rapidly. Also it allows the aid delivery efficiently.

3.6.1.6 Main and important clauses an extracted from Specifications for Highway Bridges of Japan (Part V Seismic Design) for references as below.

Extracted from Specifications for Highway Bridges of Japan (Part V Seismic Design)

CHAPTER 2 BASIC PRINCIPLES OF SEISMIC DESIGN

2.1 Fundamentals of Seismic Design

(1) In the seismic design of a bridge, the seismic performance required depending on the levels of design earthquake ground motion and the importance of the bridge shall be ensured.

(2) In seismic design, structural planning shall be developed while considering topographical, geological, and soil conditions, site conditions, local anti-tsunami plans, etc. Necessary seismic performance shall be secured in the design of individual structural members of the bridge as well as the entire bridge system.

2.2 General Matters on Seismic Design

(1) Two levels of design earthquake ground motions shall be considered in the seismic design of a bridge. The first level corresponds to an earthquake with high probability of occurrence during the bridge service life (called" Level 1 Earthquake Ground Motion" hereafter), and the second level corresponds to an earthquake with less probability of occurrence during the bridge service life but strong enough to cause critical damage (called "Level 2 Earthquake Ground Motion"). For the Level 2 Earthquake Ground Motion, two types of earthquake ground motion shall be taken into account, namely, Type I of an interplate earthquake with a large magnitude and Type II of an inland crustal earthquake.

(2) Depending on the importance of bridges such as road class, bridge functions and structural characteristics, bridges shall be classified into two groups: bridges of standard importance (Class A), and bridges of high importance (Class B). The definitions of the importance classification are specified in Clause 2.3.

(3) Seismic performances of bridges shall have the following three levels, in view of the seismic behavior of bridges:

1) Seismic Performance Level 1
Performance level of a bridge keeping its sound functions during an earthquake.

2) Seismic Performance Level 2
Performance level of a bridge sustaining limited damages during an earthquake and capable of recovery within a short period.

3) Seismic Performance Level 3
Performance level of a bridge sustaining no critical damage during an earthquake.

(4) Depending on the levels of design earthquake ground motions and the importance of bridges, the seismic design of bridges shall conform to the following.

1) Both Class A and Class B bridges shall be designed so that the Seismic Performance Level 1 is ensured when subjected to the Level 1 Earthquake Ground Motion.

2) Class A bridges shall be designed so that the Seismic Performance Level 3 is ensured when subjected to the Level 2 Earthquake Ground Motion, while Class B bridges should be designed so that the Seismic Performance Level 2 is ensured when subjected to the Level 2 Earthquake Ground Motion.

(5) Bridges shall be designed so that unseating of superstructures can be prevented even though bearing support systems are destroyed by the bridge's complicated seismic responses or ground displacement associated with lateral spreading of ground.

2.3 Classification of Importance of Bridges

The importance for both Class A bridges and Class B bridges defined in 2.2 (2) shall be classified as defined in Table 2.3.1.

Table-2.3.1 Classification of Importance of bridges

Class	Definitions
Class A bridges	Bridges other than Class B bridges
Class B bridges	· Bridges of National expressways, urban expressways, designated urban expressways, Honshu-Shikoku highways, and general national highways. · Double-section bridges and overbridges and other especially important bridges in view of regional disaster prevention plans, traffic strategy, etc. among prefectural highways and municipal roads.

CHAPTER 3 LOADS TO BE CONSIDERED IN SEISMIC DESIGN

3.1 Loads to be Considered in Seismic Design and Their Combinations

(1) The following loads shall be taken into account in the seismic design:

1) Primary loads
The primary loads defined in Clause 2.1 of Part I Common, excluding the live load and the impact, shall be considered:
a. Dead load (D)
b. Prestress force (PS)
c. Effect of creep of concrete (CR)
d. Effect of drying.shrinkage of concrete (SH)
e. Earth pressure (E)
f. Hydraulic pressure (HP)
g. Buoyancy or uplift force (U)

2) Secondary loads
Effects of earthquake (EQ)

(2) Combinations of loads shall be as follows:
Primary loads + Effects of earthquake (EQ), specified in (1)

(3) Loads and their combinations shall be determined in such manners that they cause the most adverse stresses, displacements and effects.

3.2 Effects of Earthquake

The following seismic forces shall be taken into account to determine the effects of earthquake.

(1) Inertia force due to the dead weight of the structure (called "inertia force" hereafter)
(2) Seismic earth pressure
(3) Seismic hydrodynamic pressure
(4) Effects of liquefaction and liquefaction-induced lateral spreading
(5) Ground displacement during an earthquake

[Source: Specification for Highway Bridges of Japan (Part V Seismic Design)]

3.6.2 Unsymmetrical Design/Span arrangement

3.6.2.1 If the bridge has unsymmetrical span arrangement both in cross sectional and longitudinal direction, dynamic analysis shall be done accelerating the actual maximum earthquakes' movement in the history.

3.6.3 High/very High piers

3.6.3.1 High piers reacts to earthquakes movement uniquely in most of the cases. Therefore, the bridge with piers more than 30m of height requires dynamic analysis to secure the resistance capacity against the earthquakes.

3.6.4 Bridges with Integral structure of substructure and superstructure rigidly connected.

3.6.4.1 If the bridge has long span and its substructure and superstructure are connected together rigidly, it will react uniquely to earthquake movement. Therefore, the dynamic analysis shall be executed to secure the resistance capacity against the earthquakes.

3.6.5 Combination of Complicated Conditions

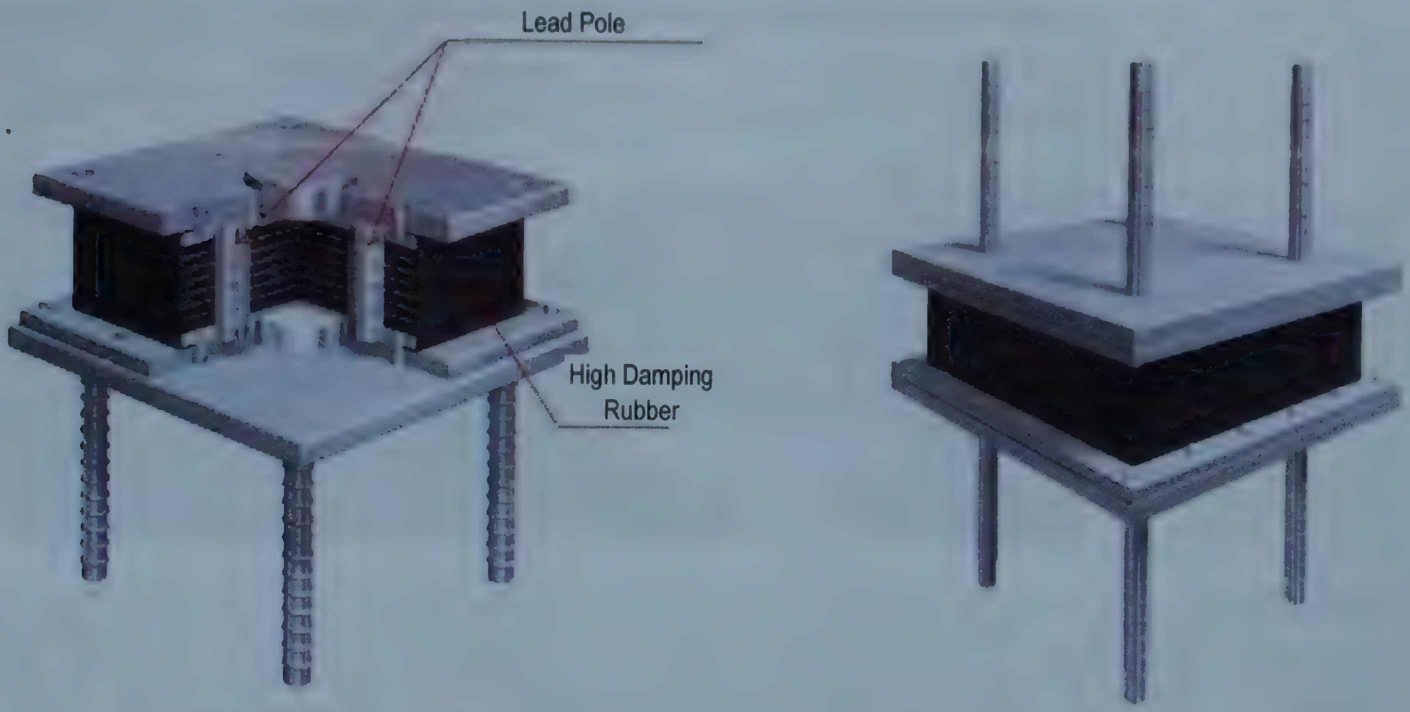
3.6.5.1 If the conditions of the dynamic analysis is multiply applied to the bridge, it will react to earthquake movement more uniquely. Combination shall be considered for the analysis.

3.6.6 Aseismic Bearing/Device

3.6.6.1 There are 2 types of aseismic bearing to sustain continuous superstructure. One is seismic isolation bearing another is horizontal force dispersing bearing. Seismic isolation bearing has more capacity to resist against the seismic force.

3.6.6.2 Both types is made of multi-layer of high elastic rubbers. Seismic isolation bearing has fat lead pole that lengthen the movement of the superstructure and damps the force. Horizontal force dispersing bearing controls the horizontal force produced by earthquake and transferred from superstructure to substructure. If the bearing capacity of a pier is weak, the force would be reduced by softer/low elastic rubber. If a pier has stronger bearing capacity, the force it distributed more with harder/high elastic rubber.

3.6.6.3 The work flow of this reinforcement works are similar to that of the bearing replacement. The details shall be referred to the clause 5.3.4.2 'Procedure'. However, aseismic bearings have higher height than the original bearings. Therefore, securing the space to install the aseismic bearings



(Seismic Isolation Bearing) (Horizontal Force Dispersing Bearing)

Figure 3-17: Example of Aseismic Bearings

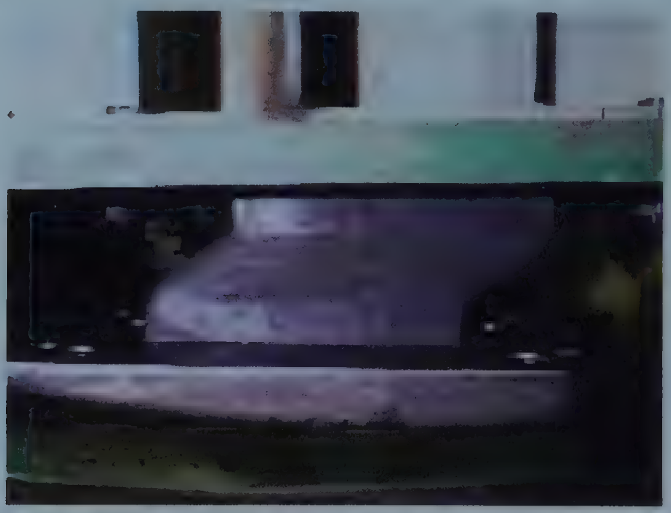


Figure 3-18: Example of Deformation of Aseismic Bearings

3.6.7 Bridge Fall Prevention Device

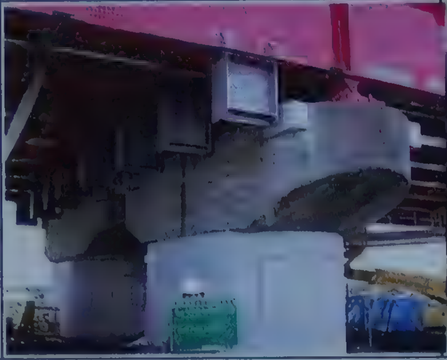
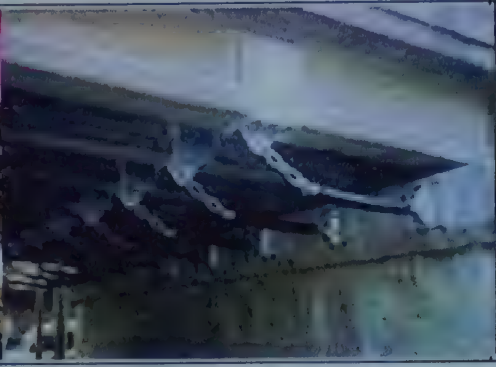
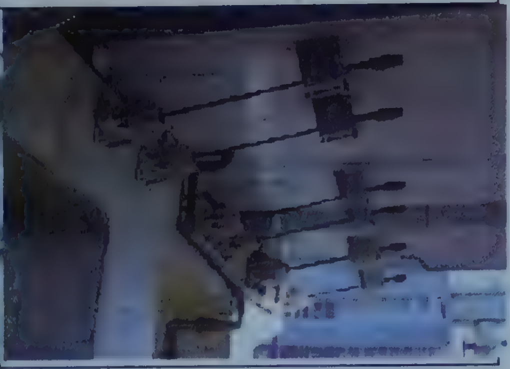
3.6.7.1 No matter how well prepared to prevent collapse of bridge, it is impossible to make the possibility of collapse 0%. There could be the historically largest earthquake/rain fall/wind giving impact to the bridge after its opening. Therefore, bridge fall prevention device is the one option to prevent the superstructure to fall from abutment/pier.

3.6.7.2 Using this mitigate the damage and reduces the amount of displacement of the superstructure. Therefore, emergency repair can be executed to secure the emergency traffic rapidly.

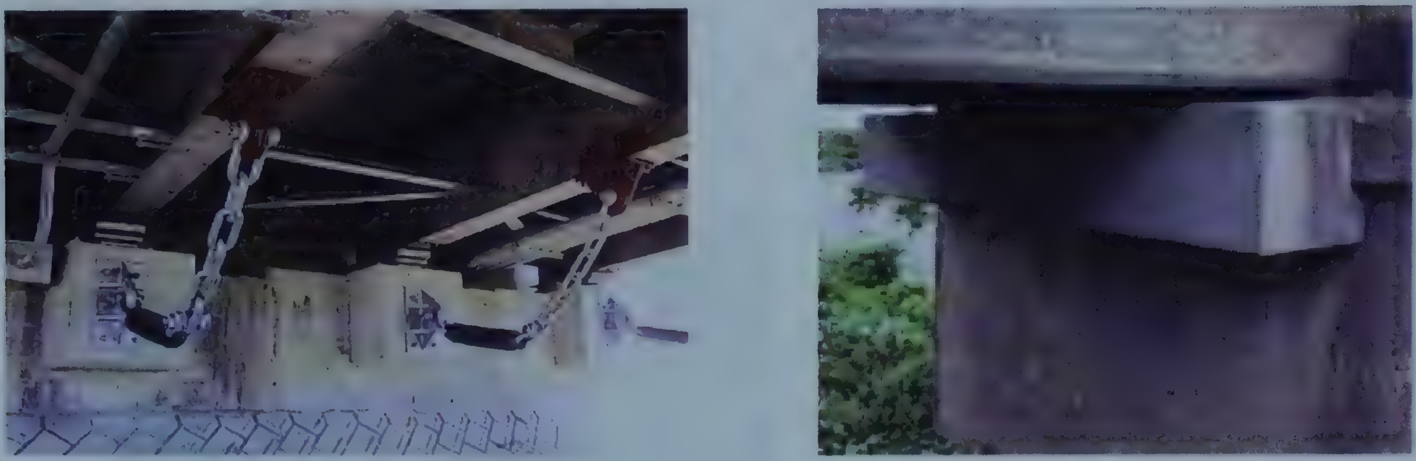
3.6.7.3 Simple beam superstructure is the most fragile structure for fall of superstructure. It can been fallen by collapse of foundation, column, and abutment wall, and also by displacement of superstructure.

3.6.7.4 The reinforcement method of preventing superstructure of falling is to limit the relative displacement of superstructure with substructure by compressive or tension force. The former is installing steel/RCC brackets to the superstructure so that the displacement of the superstructure would be limited to the location of the bracket. The latter is to connect the superstructure with substructure by cable or chain.

Table 3-13: Reinforcement Method to Prevent Fall of Superstructure

	Compressive Force	Tension Force (Cable)	Tension Force (Chain)
Outline	Steel/RCC bracket attached to superstructure	Connect superstructure with substructure by cable	Connect superstructure with substructure by chain
Image			

3.6.7.5 There is another simple method to prevent superstructure fallen is to widen the width of bridge seat both on abutment and pier. This method is often applied together with the fall prevention devices mentioned in the table below. It can be applied alone for short span bridges with small possibility of massive displacement. Images of widening of bridge seat are shown below.



(Combination with Other Device) (Widening Alone)

Figure 3-19: Example of Widening of Bridge Seat

3.7 Preparation of Next Step in Bridge Implementation Project

3.7.1 Contract Document

3.7.1.1 Contract document will be prepared in the form which suit the procurement scheme. If the scheme is PPP/PFI, it shall cover design, constitution and maintenance. If the scheme is EPC, it shall cover design and construction. If the scheme is only for construction, contract documents will be just for construction.

3.7.2 Selection construction contractor

3.7.2.1 No matter what the scheme will be, construction contractor shall be selected in this stage. Regional experience and experience in construction of similar types of mountain bridges shall be advantage in selection procedure.

CHAPTER 4 CONSTRUCTION

4.1 General

4.1.1 General

4.1.1.1 Mountain bridges construction is unique by its construction site, which mean that it is very difficult to generalize. Therefore not only the knowledge but also experience is necessary to perform well as a selected contractor or supervising engineer.

4.1.1.2 Natural environment especially the climate condition in mountainous region is very severe and could effect to the construction conditions and schedule. If the site is located in the region where it is fragile to natural disaster, there is high risk of delay of construction period. Therefore schedule control of the each works as well as the entire project is more critical compared to the bridge projects in flat terrain.

4.1.2 Scope

4.1.2.1 Scope of this guideline for construction of mountain bridges is bridges with 6m or longer of its length and located on the national/state highways/expressways.

4.1.2.2 Moreover, the bridge is located not only in mountain region but also on the mountainous and steep terrain which has the cross sectional slope percentage 25% and more.

4.1.2.3 There are terrain less than 25% of cross sectional slope in mountain region as well, for instance alluvial basin. There is normally large area of plane in these regions. This guide line can be used as reference for the bridge in plane/rolling terrain in mountainous region. However, these bridges usually have similar characteristics with bridge in plain region more than with mountain bridges. Therefore, responsible bridge engineer shall select the contents to follow for these kinds of bridges

4.1.3 Objective

4.1.3.1 This guideline aims to supply additional but necessary information of construction of mountain bridges which are not mentioned in already issued IRC Codes for realising safe/efficient/economical implementation, long service life span, lower life cycle cost, resilient to natural disasters. It shall contribute for the regional development in mountainous region in India.

4.1.3.2 Not like preparation or designing of bridges, there is not any integrated IRC Code for bridge construction. The most probably exact publication to follow and add the information is 'Pocketbook for Bridge Engineers, 2000 (First Revision)'. Therefore, the description in this guideline is basically the additional but necessary information to 'Pocketbook for Bridge Engineers, 2000 (First Revision)'.

4.1.3.3 Subchapter 4.2 describes the additional data collection required for the review of detailed design before the construction to prevent/minimize considerable increase of construction cost due to appearance of unforeseen conditions. No matter how much in details the detailed designed is performed, there are always unforeseen conditions which would appear just before or during the construction stage. The mountainous and steep terrain, this tendency is higher than bridges in plain terrain. It contains subchapters, such as data collection, survey and investigation, revision of detailed design.

4.1.3.4 Subchapter 4.3 explains the construction of bridge foundation on slopes by types of foundation including deep foundation which is common technology in Japan for the foundation on slopes. There is presentation of one of the most efficient construction methods, so called 'Bamboo Split Shaped Retaining Method'

4.1.3.5 Subchapter 4.4 explains the construction of substructure on slopes of abutment and pier including high/very high piers which are common technology in Japan for the mountain bridges on highway/expressway on mountainous and steep terrain. There are presentation of some efficient construction methods which have wide experience till present.

4.1.3.6 Subchapter 4.5 explains the construction of superstructure on slopes by types of superstructure including several technologies which are applied frequently for mountain bridges on highway/expressway on mountainous and steep terrain. There are description of 'Steel Orthotropic Deck', 'Composite Superstructure', 'Weathering/Weatherproof Steel', and 'Multipolar Three Dimensional Frame with RC Deck'

4.1.3.7 Subchapter 4.6 treats river protection. Mountain bridges have to face rapid river/flood flow with considerable size of debris. This flow damages not only bridge structure but also the river bank and protection structure which could end up damaging the bridge seriously.

4.1.3.8 Subchapter 4.7 explains the required report during the construction and necessary for completion of the construction stage.

4.2 Review of detailed design

4.2.1 General

4.2.1.1 Results of detailed design shall be reviewed and if necessary it shall be revised before construction. Necessity of review is quite different by scheme of the project.

4.2.1.2 In recent years, detailed design is performed by contractor in India. All of the procurement schemes of highways/expressways is defined that the same contractor of the project is responsible of detailed design and construction, such schemes are BOT, VGF, Annuity model, HAM, and EPC. Therefore, review of detailed design is done by the project contractor itself. It is done to make the design more efficiently for the project.

4.2.1.3 There is only one opportunity of review by other parties apart from the member of the projects contractor, such as the procedure of approval of detailed design by project owner which is government authority. This process shall be done by plural responsible and experienced engineers to prevent sever constraints of construction, such as errors, unnecessary work items/materials/equipment, risk of serious accident, risk of severe destruction of natural environment, etc.

4.2.1.4 If the project is procured in conventional method, such as design and construction in different contract, Construction contractor is usually very eager to review the result of detailed design to utilize their favourable method or know-how to the construction. Therefore, the project efficiency is usually improved in this stage. This is the one of the advantages of conventional procurement. However, it requires longer time in governmental administration procedure.

4.2.1.5 Mountain bridges are usually far from the office of design firm and self-revision by design firm is limited in the aspect of unique conditions of the area and the site. There could be some important/critical facts that affects the construction efficiency considerably missing in the results of detailed design. Therefore, review shall be done by plural responsible and experienced engineer to secure there is not any critical condition missing in the results.

4.2.1.6 Particularly, the new critical condition for construction method and schedule could be found out after the commencement of the project. It shall be confirmed as early as possible and referred to the revision to improve the efficiency of the project technically, environmentally, economically, and financially.

4.2.1.7 Subchapter 4.2 describes following 3 contents, such as Data Collection, Survey and Investigation, and Revision of Detailed Design

4.2.2 Data Collection

4.2.2.1 General

(1) Theoretically there shall be no data collection required for construction after the execution of detailed design and approved by project implementation authority. However, it is usual that there are always new findings which affects the project progress or efficiency considerably. Therefore, responsible bridge engineer shall make sure that there is not any important/critical data/information missing in the results of detailed design.

(2) Mountain bridges are located in severer natural environment than that of bridge in plain/rolling terrain. Moreover, sub-soil, river/flood flow, slope, climate, and ecosystem (flora and fauna) are unique by bridge location, even there are similarity among the nearby bridges. Therefore, responsible bridge engineer shall make sure that all the data/information necessary is collected for every bridges in the project.

(3) Ignoring the fact which affects the bridge could lead to the huge increase of construction cost and delay of construction period. Moreover, it could lead to decrease of the life span or increase maintenance cost drastically, which results in spending enormous amount of repairing the bridge and increase of LCC of the bridge. Since Indian governmental authority is applying financial scheme which shift financial risk to private sector, finding critical but unrevealed facts as early stage as possible prevents or minimize these risks become apparent.

4.2.2.2 Topographic data

(1) Since there are detailed topographic data as a 3D CAD data in recent years, review shall be done to secure the accuracy of the data already collected in detailed design. Since the surrounding terrain of the mountain bridge is rough, there shall be mismatches with the 3D CAD data and the real terrain. If the mismatches are large, it could influence the design or construction planning.

(2) Uneven surface, such as swell, pocket, gap, drop-off, artificial structures (retaining walls, ditch, etc.) shall be carefully examine. If it is missing in the 3D CAD data, footing or even foundation could appear above the ground surface/riverbed and it can affect the design. On the other hand, the cover could be too thick and construction would be delayed. It could occur much more at the site of mountain bridges than at the site of the bridges in other smoother terrain.

(3) If the bridge is crossing the river, and the flood flow is rapid and massive, the river bank and bed could change and main flow could shift after the topographic survey. Moreover, slope could slide, vegetation could change, new structure could been built. Mountainous and steep terrain is very fragile in some location, therefore, there shall be plural changes of the surrounding terrain of the bridge site. It could give negative impact to bridge design and construction planning. Changes from topographic survey shall not be overlooked and reflected to the latest topographic data of the site.

4.2.2.3 River and catchment area related data

(1) Review of the hydrological design condition shall be done to secure the durability of the bridge and safety in construction as well as in operation. The error in setting the design condition could affect the design, obstruct the construction, and damage the bridge. It would increase both construction and maintenance/repair cost, which end up with very high LCC.

(2) What responsible bridge and hydrological engineer has to be very careful is the change of hydrological condition from previous study (detailed design stage). Specifically the Himalayan Mountains are covered with very fragile slopes, even one monsoon season can change the condition of the catchment area and river flow of the bridge. If the river flow of the crossing point has been changes considerably, design river cross section shall be revised according to the latest flow, including HWL, and scouring depth, and etc. It could affect to the design of foundation and substructure.

(3) As is written in the previous clause, if the bridge is crossing the river, and the flood flow is rapid and massive, the river bank and bed could change and main flow could shift after the site survey in detailed design stage. Light debris flow could rise the riverbed, massive flood flow could dig the river cross section deeper. These changes shall be referred not only to topographic map but also to river/flood flow condition including scour depth and characteristics.

(4) India has wide area of mountainous region with very fragile slope, represented by the Himalayan which shifts as much as 7cm annual in some locations. Therefore, river/flood condition shall be reviewed and revised till the last moment if necessary to reflect the site condition to the design.

4.2.2.4 Sub-soil strata data

(1) Additional sub-soil strata data shall be collected if the responsible bridge or geotechnical engineer recognize that there are missing data to define the bearing layer condition, such as degree and direction of the slope of bearing layer. The foundation of the bridge shall be placed to bearing layer no matter what kind of the foundation is applied.

(2) If there are considerable change of riverbed due to flood/debris flow after the detailed design, physical condition of the riverbed down to the souring depth shall be recollected to revise the scouring depth of the foundation/substructure. Since the rivers in Himalayan Mountains have flood/debris flow frequently, the change shall be checked carefully in any cases.

(3) New findings of land slide could be the reason to collect additional data for sub-soil strata to define the affected area. Bridge shall not be located inside of the land slide affected area.

4.2.2.5 Climate data

(1) Additional climate data shall be collected when there are historical record breaking phenomenon occurred at the site or the surrounding region after the latest condition is fixed. Specifically the earthquake, rainfall and wind which influence the design considerably.

(2) Responsible bridge engineer shall be very careful that the record breaking phenomenon of recent year is surely referred to the design condition. There could be time lag that the data is referred to the official record provided to the design consultant in detailed design stage. Updating the climate data is very critical to the design. It would increase both construction and maintenance/repair cost, which end up with very high LCC.

4.2.2.6 Traffic data

(1) Additional traffic data shall be collected when there is considerable change in community or region socially or commercially, such as massive development/clearance has newly started. However, it is rare that additional data is required in this stage, since the plan of the social or commercial development/clearance plan is usually included in the traffic demand forecast and does not influence the current traffic volume.

4.2.2.7 Procurement data

(1) Additional procurement data of material, equipment, and facility is required to be collected in this stage, since these data is based on the standard unit price issued by governmental authority and there are some considerable gap with the real unit price. It shall be well collected to define the construction cost estimate for construction contractor. Most of the time, the design firm is not able to estimate the construction cost in details as the construction contractor requires. It could affect total construction plan for the bridge if the gap is very large to make the profit of the contractor.

(2) If the gap between standard unit price and the real unit price is too large, the contractor used to claim to the project owner for the amendment of the contract. However, in recent years the Indian governmental authority applies the financial scheme with less opportunity of amendment, such as BOT, VGF, Annuity model, HAM, and EPC. Therefore the construction contractor shall collect the procurement data before the submittal of the bid document and be careful of setting the real unit price and estimate total bid price so that the budget of the project does not becomes short.

(3) However, due to the severe natural conditions which mountain bridges needs to face, there are always new findings after the contract amount of the project is fixed or construction has commenced, both parties project owner and construction contractor shall be flexible to make amendment of the contract when the new findings are very serious. Failing of the amendment could lead to the contractor's cancelation of contract or insufficient quality of construction and lead to spend extra cost for the construction or maintenance.

4.2.3 Survey and Investigation

4.2.3.1 General

(1) The additional survey and investigation, especially at the site, is required due to the reasons mentioned in previous clause 4.2.2 'Data Collection' to reduce and minimize the risk of appearance of new findings after the commencement of actual construction works. It would also minimize the construction and maintenance cost.

(2) Mountain bridges are located in complicated terrain and it is very difficult to collect all of the exiting data to duplicate the site completely by setting the design conditions. However, unique condition of each site is only obtained by site survey and investigation. Executing additional site survey/investigation would delay the entire construction schedule and increase the preparation cost. However, the delay and cost increase is not large and also could shorten and reduce them in total. Therefore, not only the responsible bridge engineer but also project manager, responsible personnel of project owner/government shall not hesitate to execute additional survey/investigation when there is lack of necessary data/information.

(3) Ignoring the fact which affects the bridge could lead to the huge increase of construction cost and delay of construction period. Moreover, it could lead to decrease of the life span or increase maintenance cost drastically, which results in spending enormous amount of repairing the bridge and increase of LCC of the bridge. Since Indian governmental authority is applying financial scheme which shift financial risk to private sector, finding critical but unrevealed facts as early stage as possible prevents or minimize these risks become apparent.

4.2.3.2 Topographic Survey

(1) Additional topographic survey shall be done when there is mismatches between 3D CAD data and the real terrain, specifically for uneven surfaces, such as swell, pocket, gap, drop-off, artificial structures (retaining walls, ditch, etc.) along the alignment and within the area or Right of Way (ROW) shall be collected additionally. It prevents the cover of the footing of the substructure too thick or appearing above the ground surface. Both would result in design change, increase of construction cost and delay of construction period. Since mountain bridges have rough terrain, insufficient height of the footing often occurred and recognized during the actual construction works. Close review and additional survey prevents and minimize this risk considerably.

(2) River cross section is the other item with considerable mismatches between 3D CAD data and the real terrain, since massive/rapid flood flow, debris flow, washes the riverbed, shifts the main flow, and alleviates debris. The considerable change from the previous topographic survey execution, requires additional topographic survey to correct the mismatch of the 3D CAD data and the real terrain.

4.2.3.3 River and catchment area investigation

(1) Change of the river cross section affects not only the riverbed level but also the scour depth. Characteristics of the riverbed material and sub-soil down to the scouring depth shall be collected and referred to the review/revision of scour depth.

(2) If the entire or majority of the catchment area is affected by natural disaster after the previous hydrological survey, the site survey shall be additionally executed. It could change the design river discharge, HWL, and etc. and affect to the design. Ignoring the change of the catchment area and hydrological conditions without executing additional survey could damage the bridge severely after opening the bridge and increase the maintenance/repair cost, or shorten the service life span. Additional survey prevents and minimizes these risks and reduce the LCC.

4.2.3.4 Boring survey

(1) Additional boring survey shall be executed is the level of top surface of the bearing layer is uncertain. Every pile shall be penetrated to the bearing layer, and the entire bottom surface shall be placed on the bearing layer. If the design result is not satisfying this condition, it shall be modified by applying step footing or various length of piles. To secure this requirement, the details of slope direction and gradient shall be revealed clearly. Therefore, additional boring survey is often necessary before and during construction of mountain bridges.

(2) Additional survey and test for riverbed material if there is considerable change on the riverbed after the previous boring survey to review the scour depth and revise the design if necessary. Riverbed of mountain bridges are very fragile and changes very frequently by flood/debris. The influence of the change shall be checked if the change is considerable.

(3) If there is new finding for land slide at the bridge site, it shall be confirmed that the bridge is out of the affected area. If additional boring survey is required to check the status, it shall be executed. Responsible bridge engineer shall have close communication with geologist/geotechnical engineer to find out the necessity of additional boring survey.

4.2.3.5 Procurement Investigation

(1) Mountain bridges are usually in the mountainous region and with rich resource of aggregate. However, it is often very far from material/manufacture factories and requires high cost of transportation. Actual price of procurement is only possible to obtain by site survey and quotation from the supplier providing the conditions in details. Collecting this detailed conditions, the construction contractor has to execute the site reconnaissance and set the condition or modify the condition from detailed design.

(2) Construction contractor shall survey the access route for heavy construction machinery to the site, specifically if the existing road is used to access the site. In many cases with mountain bridges, the existing road to the site are local road with narrow width and curve with small radius. Some of the machinery cannot access the site using the exiting road. Machinery has to be changed to smaller one or the access route has to be developed.

4.2.4 Revision of Detailed Design

4.2.4.1 Structural Aspect

(1) It is not common that the structural system is reviewed and modified/revised entirely in this stage of the bridge implementation after the detailed design result is approved. However, whenever new findings which affects the entire bridge is found before the commencement and during the construction, the basic structure shall be reviewed and revised. The example of findings are such as land slide, large shift of main river flow, or archaeological monument, etc. which required alignment change centre line of road, or shift of substructure.

4.2.4.2 Procurement materials, equipment, facilities, labour

(1) The details of procurement of the material, equipment, facilities, and labour are usually not mentioned in detailed design report or bid document of construction, or other contract documents of bridge implementation. However, it is described in the construction plans to estimate the construction cost. It is usually not so accurate enough to immediately commence the construction works. Therefore, it shall be revised with the additionally collected data/information including the site reconnaissance and investigation.

(2) Review of procurement is linked to review of construction method, plan, schedule, and cost estimates. If one of the item is reviewed and revised, the other items shall be revised accordingly.

(3) Mountain bridges have unique condition of procurement by region. The cost is always the most important factor to select and construct the supply chain of the construction. However, if the supplier is too far from the site, procurement schedule shall be considered. Frequent delay would affect the construction schedule as well as cost.

4.2.4.3 Construction Method

(1) Basic construction method of bridge main members, such as foundation, substructure, and superstructure are seldom changed in this stage of bridge implementation after detailed design in approved. However, minor revision could be possible due to change of construction equipment or etc. it could happen more in the site which is far from urban area where the equipment is procured. Mountain bridges are often located in the isolated area far from urban area and it have high possibility of minor changes.

4.2.4.4 Construction Planning and Scheduling

(1) Construction planning and scheduling are the items which requires considerable revision in this stage if the construction contractor is not responsible for the detailed design, such as conventional method of implementation. It is because the construction plan is prepared by design firm which is responsible for detailed design to estimate the construction cost and often does not have accuracy to start construction immediately. Construction contractor shall include the method with rich experience to speed up the construction, improve the quality, and minimise the construction cost.

(2) However, since detailed design is performed by construction contractor in the new schemes in India, such as BOT, HAM, EPC and etc., there shall not be many points to revise by construction contractor themselves. Therefore, brief review by the third party could be useful for these cases.

4.2.4.5 Cost Estimate

(1) Cost estimate shall be revised after the all of the other items are revised, based on the revision made for the other items. This procedure is same with bridges in other terrain considering that the revision of the unit cost is done in procurement revision.

4.2.4.6 Environmental Countermeasure

(1) Environmental countermeasures are described in Environmental Impact Assessment and it shall be followed all the period of construction stage. Mountain bridges are often in the non-urban area with fragile nature, therefore even short period of malfunction of the countermeasures could give severe impact to the object which shall be protected. Construction contractor shall review the countermeasure in details and be sure that it would not leave the negative impact to the nature in surrounding area, specifically is there exists species in danger both flora and fauna.

(2) Monitoring method is another important part of protecting fragile nature around mountain bridges. Construction contractor are not permitted to change the entire monitoring system usually but they are allowed modify the system to make it more effective and efficient in most of the cases. There is an opportunity that the review been done.

4.2.4.7 Financial Procurement

(1) Since Indian government is actively applying the contract scheme which requires financial procurement by the contractor, such as BOT, VGF, Annuity Model, and HAM, for road construction, review of financial procurement shall be reviewed until the very last moment before the construction begins, or even during the construction. The project's financial manager shall always seek to minimize financial risk and financial cost throughout the contract period.

4.3 Foundation

4.3.1 General

4.3.1.1 Foundation is the fundamental member of the bridge which sustain the entire bridge above of it. However, since it is constructed inside the sub-soil below the ground surface, it often happens that the design documents has considerable mismatches with the actual reality of the sub-soil condition and derives unexpected situation, such as delay, additional cost, insufficient quality of structure, etc.

4.3.1.2 To prevent or mitigate these risks mentioned above, frequent troubles and its countermeasures shall be well known to the responsible bridge and geotechnical engineers for them to be ready from the preparation to completion during the foundation construction. It is also very important to record the actual condition of the sub-soil data which was revealed during the construction to recognize the actual condition, mismatches with design document, and select the countermeasures.

4.3.1.3 Mountain bridges are usually located on the very complicated sub-soil which has higher risks of mismatches between the design documents and actual reality. Moreover, the sub-soil is prone to have large size gravel which requires special treatment in excavation. Also the underground water level rises rapidly dependent upon the rain at and upstream of the bridge site. Having the knowledge and the experience of foundation construction in mountain region is essential to mitigate the risks above and satisfy the quality of actual construction work.

4.3.2 Spread Foundation

4.3.2.1 Spread Foundation on Flat Bearing Layer

(1) If the substructure has flat bearing layer, it is basically the same with the bridges in other terrain, the conditions of bearing layer shall be confirmed by sight observation and flat-plate loading test if necessary, before casting the levelling concrete to secure the bearing capacity which is indicated in design document.

(2) This confirmation procedure shall be done at every location of substructures. When the bearing layer differs by substructure or considerable cracks are recognized on the surface of the bearing rocks, site loading test shall be carefully done to secure the bearing capacity in design document. If it is found out that the actual bearing capacity is less than the design document, the design has to be reviewed whether the layer could bear the load of the bridge and maintain the stability of the entire bridge.

4.3.2.2 Spread Foundation on inclined Bearing Layer

(1) If the substructure is placed on inclined bearing layer, it is important to confirm the actual conditions of it by sight observation. Since the design budget and period is limited, it often occurs that the actual depth of the layer, degree and direction of incline of the layer are different from the design document. Therefore, when the difference is considerable the revision of the shape of the spread foundation shall be done to secure the solid penetration to the bearing layer. Re-bar arrangement shall be reviewed accordingly.

(2) If the degree of the incline of the bearing layer is steep, the spread foundation is usually designed with steps or replacement of ground soil. In case of the revision of the spread foundation, shape and re-bar arrangement shall be revised to secure the solid penetration to the bearing layer.

(3) Mountain bridges with substructures on spread foundation tends to be located on the inclined bearing layer. It is also have high risk of the mismatch between the actual condition, such as depth, degree and direction of slope, bearing capacity, and that of design document. Therefore the responsible bridge/geotechnical engineer shall be ready to investigate/confirm the actual condition and review/revise the design results to fulfil the necessary structural requirements.

4.3.3 Pile Foundation

4.3.3.1 General

(1) Pile foundation are placed on the bearing layer which is located under the ground surface with considerable depth. Therefore, the conditions, such as depth, degree and direction of slope, bearing capacity, are determined and calculated from the result of boring survey. However, since it is located in the deep underground, it is not able to confirm the actual situation by sight observation as spread foundation.

(2) Construction of pile foundation is dependent on the soil condition of sub-soil above the bearing layer as well. If there is large size of the gravel, most of the precast/steel piles are not able to dig in and be penetrated to the bearing layer for instance. The depth of the bearing layer affects to the type of the pile foundation as well. Most of the precast PSC piles are difficult to reach the depth more than 40m. Steel piles and bored piles are usually up to 60m, if the pile needs the length more than 60m, there are limited method of foundation types and construction method for the bridge with such a deep bearing layer.

(3) Since the condition of sub-soil strata cannot be confirmed by actual sight observation and the budget of the design is limited, it often happens the mismatch between actual condition and results of the design. Therefore, the actual condition shall be confirmed for every pile prior to excavation or during excavation. Sudden drop or uplift of the bearing layer, massive gravel appearance in intermediate layer cannot be observed only by the boring survey in surrounding points. Since every pile shall be penetrated to bearing layer in general, adjustment of length, excavation method, additional equipment, and etc. shall be applied to secure the bearing capacity of all the piles of substructure/bridge.

(4) Since mountain bridges are often planned above the complicated sub-soil strata, responsible bridge engineer shall be ready to review/revise the design to secure the bearing capacity of all the piles with close communication and cooperative relationship with geotechnical engineer.

4.3.3.2 If the substructure construction site is located below the river level, there is limitation of season in constructing the foundation. The rainy season shall be avoided even it is urgent to implement the bridge. It is very difficult to secure the safety and also prevent the construction cost from increasing. Monitoring the precipitation and discharge shall be done anytime of the year to secure the safety.

4.3.3.3 Pile Foundation on Flat Bearing Layer

(1) If the substructure has flat bearing layer, it is basically the same with the bridges in other terrain, the conditions of bearing layer or intermediate layer shall be confirmed during the excavation and with test piles excavation and loading test if necessary, before excavating other piles to secure the bearing capacity which is indicated in design document.

(2) Moreover, quality of materials, the accuracy of the excavation, location, incline, and sufficiency of bearing capacity of pile end shall be checked and recorded during pile foundation construction.

4.3.3.4 Pile Foundation on inclined Bearing Layer

(1) If the bearing layer is estimated to be inclined in design stage, it shall be confirmed the degree and the direction of the bearing layer before the excavation of the pile by boring survey. However, to be certain for all the piles, boring survey shall be done for every pile and it is not realistic to execute it. Therefore, test pile result and recognition of the condition during the excavation and refer to the construction method is very important to secure the accuracy and the bearing capacity of the piles and the entire foundation.

(2) When the drop and the uplift of the bearing layer is recognized in excavation, adjustment of the length shall be done to secure the solid penetration of the end and its bearing capacity. The change shall be reviewed that the pile itself is stable and necessary revision shall be done if necessary. The adjustment method is different by the type of pile. The physical characteristics depends on the adjustment method. Some of the method is mentioned below in the description by types of pile.

4.3.3.5 Cast-in-situ piles

(1) There are mainly three methods of excavation/construction of cast-in-situ piles, such as All-casing method, Reverse Circulation method, and Earth-drilling method. Among these three methods, Reverse Circulation method requires impermeability of all the layers of sub-soil strata, both intermediate layers and the layer very near to the surface, to maintain the wall of excavated hole until the casting of the concrete from collapse. Earth-drilling method also requires impermeability of intermediate layers 2 to 4m below the surface. However, it is very difficult to have impermeability of all the intermediate layers in mountain regions since there are gravels with large diameter. Therefore, this guideline does not treat Reverse Circulation method and earth-drilling method.

(2) All-casing method

i) All-casing method is the excavation and casting concrete method of pile foundation which requires full length of steel casing to protect the wall of excavated hole until the casting of the concrete from collapse. Steel casing are penetrated into the sub-soil whether by vibration or rotation. Capacity of excavation is higher by rotation. If there is a gravel with its diameter more than 100mm, or the pile length is more than 60m, it shall be excavated by rotation.

ii) When mountain bridges on mountainous and steep terrain requires pile foundation, it rarely requires piles of more than 60m in length. However, it often requires excavation of the layer which contains gravel with its diameter more than 100mm. Therefore it is preferred to apply the excavation method by rotation. The excavation method by rotation are able to apply larger diameter for the pile, vibration method is generally up to 2000mm and rotation method is up to 3000mm. Therefore, if there is new findings of large gravel of more than 100mm in the sub-soil strata, there shall be a study to apply rotation method in the revision.

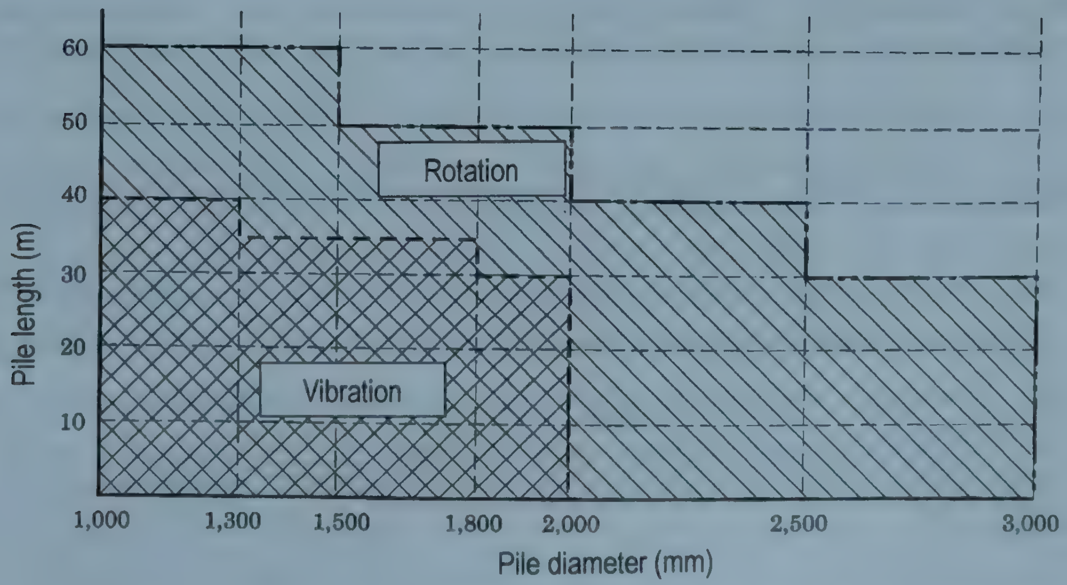


Figure 4-1: Pile Length and Diameter Relation by Excavation Method

(Source: Handbook of Pile Construction, Japan Road Association)

iii) Moreover, the excavating machine is larger and occupy larger area for vibration method than rotation method. It indicates that if the vibration method is applied, there requires more flat land to be developed for the excavation with more negative impact to the nature of the site and its surrounding area. Therefore, rotation method have more advantages for mountain bridges.

4.3.3.6 Precast/steel piles

(1) There are many type of prefabricated pile which is applied for bridge foundation. Types are divided by Material, construction method. The materials are steel pipe, pre-tensioned prestressed concrete, steel pipe filled with concrete, reinforced cement concrete. The construction methods are driven (vibration or striking by hummer) piles, inner excavation piles, pre-boring piles, soil-cement filling methods.

(2) These piles are usually applied when there are more than certain numbers of piles with same specification, length, diameter, material specification. To secure these conditions, both grand and bearing layer needs to be flat, or at least form mile slopes in parallel. However, mountain bridges are very difficult to have these conditions, therefore prefabricated piles are rarely applied for mountain bridges on mountainous and steel terrain. This guide line is for mountain bridges, therefore there is not detailed description on these types of piles. Details of these piles are described in IRC 078:2014 focusing investigation and design matters mainly.

4.3.4 Deep Foundation

(1) Deep foundation is developed with manual excavation without heavy machinery for excavation. Therefore it only requires very small flat area on the ground surface. However, small machinery, such as small micro/mini excavator or clamshell bucket, is applied. It only requires square area with diameter of the pile for manual excavation.

(2) The biggest advantage of this type of foundation is that it can observe every sub-soil layers by sight and confirm its condition. Moreover, it also confirms the bearing capacity of the bearing layer and secures the solid penetration of foundation to the bearing layer.

(3) Construction of the deep foundation is usually executed in very limited area of construction site in the middle of the steep slope. Therefore, it requires its construction plan and schedule more in details than for other type of foundation. The flexible change of the construction plan and schedule is limited due to the restricted area of construction site. The construction site is often isolated and far from the supplying storage. This fact is another reason that the flexibility is limited during construction of deep foundation. Recording the condition and the progress, and adjustment by modification is very critical in construction of this type of foundation.

(4) Securing the safety inside the hole is one of the most important point to take care of in the construction of deep foundation. Retaining wall of the hole, drainage of rainwater and underground water, work with height security are the most important works for securing the security. If the construction requires human labour inside the hole, this is very critical works for the entire construction of foundation. Therefore, automated excavation is spreading to improve the safety of the workers at the site.

(5) Underground water shall be shielded or drained from the hole to secure the constructability and prevent the collapse of the wall. It can be shielded by liner plates if the volume or the water coming out of the wall is limited. Grouting waterproofing material to the surrounding sub-soil or draining at the deep well close to the hole is one of the alternatives of drainage method. However, the impact shall be monitored to the surrounding area especially the depression and decrease of water volume of shallow wells.

(6) If human labour needs to go inside the hole, the sufficient volume of fresh air shall be send to the bottom of the hole. It shall exceed 50m³/min and 10m³/person. There could be poisonous gap or blast during the excavation, therefore there shall be gas detector installed at the working location in the hole. There shall be an evacuation plan prepared and well known to the all labour.

(7) To prevent the collapse of the top end of the wall, there shall be closed ring shaped concrete placed at the edge of the top end. If the diameter is large or requires solid structure, the concrete shall be reinforced by re-bars as shown on the picture below. It is called 'guide-wall' in Japanese construction field.

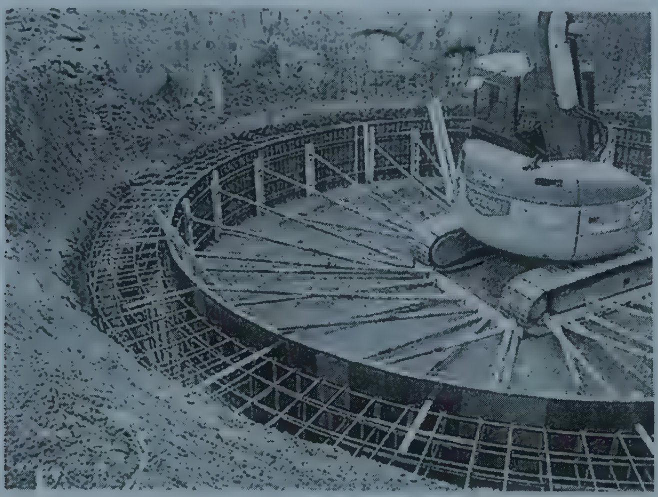


Figure 4-2: Example of 'Guide-Wall'

(Source: Handbook of Deep Foundation Construction, Japan Road Association)

(8) There are three excavation methods, by human labour, by excavation machinery, and by blasting. If the diameter is larger than 5m, excavation machinery shall be applied to improve the efficiency. If the intermediate layer has very large gravel, or is thin but with very hard rock, blasting method can be applied. However, it shall be minimized to prevent the sub-soil strata from getting loose or collapse of retaining structure of the wall of the hole.

(9) When the excavation and the sight recognition of bearing layer is completed, flat-plate loading test, or other test equivalent, shall be done to confirm the bearing capacity of the layer.

(10) The bottom surface of the excavated hole shall be immediately covered by levelling concrete if the layer has possibility of softening, such as very solid clay, tuff, or fault fracture zone.

(11) Details which shall be considered in construction of retaining wall of deep foundation is demonstrated in 'Handbook of Deep Foundation Construction, Japan Road Association'. It is extracted as bellow.

Extracted from Handbook of Deep Foundation Construction, Japan Road Association

CHAPTER IV Construction

4. Retaining Structure

4-1 Type of Retaining Structure

There are three type of retaining structure for deep foundation, liner plate, mortar lining and shotcrete. Retaining wall with liner plates places divided liner plates to edge of the circle cross section and connects together to make it as a ring structure. Retaining wall with mortar lining or shotcrete is installed by putting cement mortar or concrete on the entire surface of excavated hole. Retaining wall is not removed throughout the construction period unless the subsoil is very hard clay or hard rock which has very scarce possibility of collapse.

Retaining wall with liner plates is easy to install by bolts and very reliable structure based on the past application. Therefore, it is applied for many kinds of subsoil conditions.

On the other hand, retaining wall with mortar lining (Shot mortar or casting mortar) and shotcrete are applied to the subsoil condition which has sufficient stability until the mortar or cement concrete hardens and obtains the function of retaining wall. It shall not have underground water coming out from the wall which would be a distraction for construction.

There could be a combination of several types of retaining wall applied for a single hole of deep foundation. Layers near the ground level, such as slope talus and etc., which does not have enough stability could be retained by lining plates. Stable layers far from the ground can be retained whether by mortar lining or shotcrete. (See Figure below)

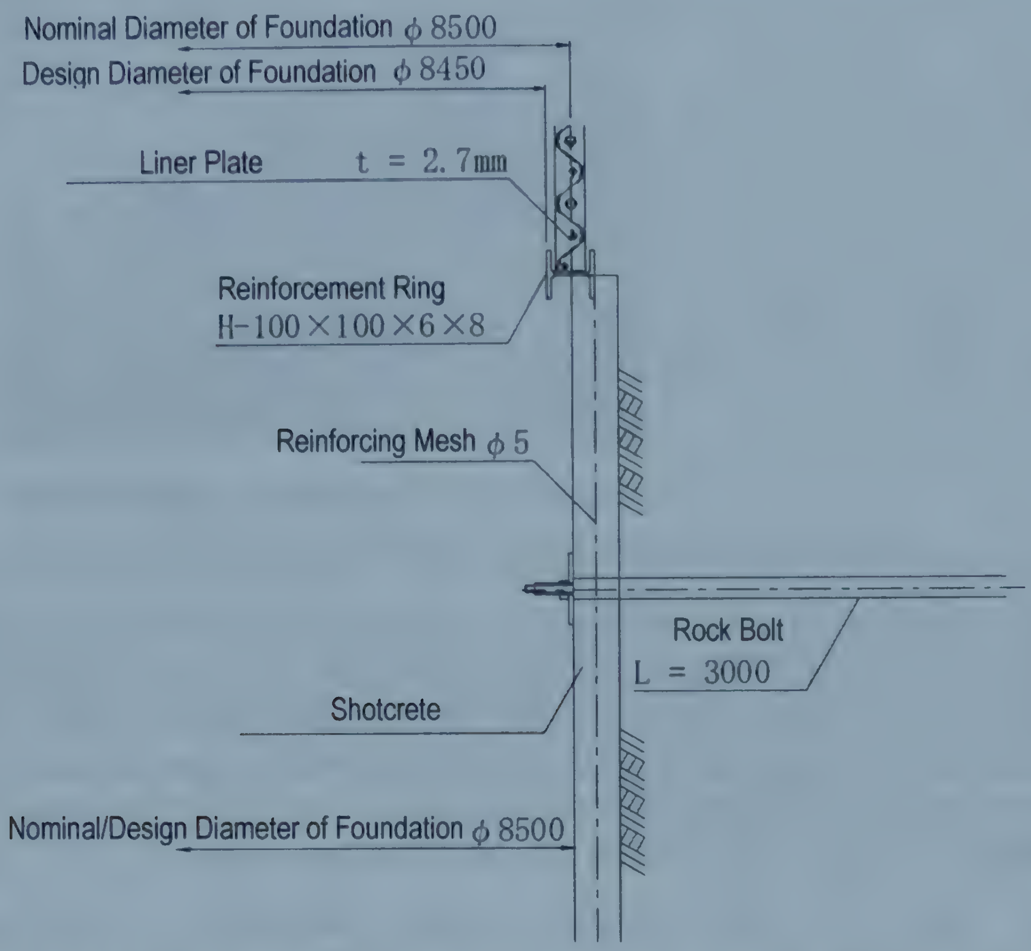

Nominal Diameter of Foundation ϕ 8500
Design Diameter of Foundation ϕ 8450
Liner Plate t = 2.7mm
Reinforcement Ring
H-100×100×6×8
Reinforcing Mesh ϕ 5
Rock Bolt
L = 3000
Shotcrete
Nominal/Design Diameter of Foundation ϕ 8500

Figure IV.4.1 Example of Retaining Wall (Combination of Liner Plate and Shotcrete)

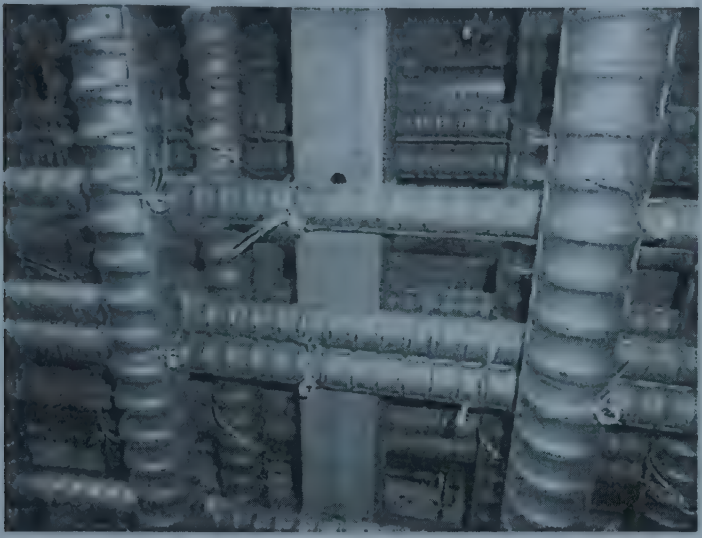
Mortar shall be injected to the entire gap between liner plate and surface of hole of deep foundation in order to transfer the load between subsoil and foundation. Retaining wall with liner plates are applied to the unstable subsoil layer, such as slope talus and etc., there shall be special care in excavating the subsoil layers which liner plate structure is applied to prevent the layers from loosening. It also requires dense injection of mortar to the gap between liner plate and deep foundation in order to secure the horizontal resistance from subsoil layers.

5. Rebar arrangement

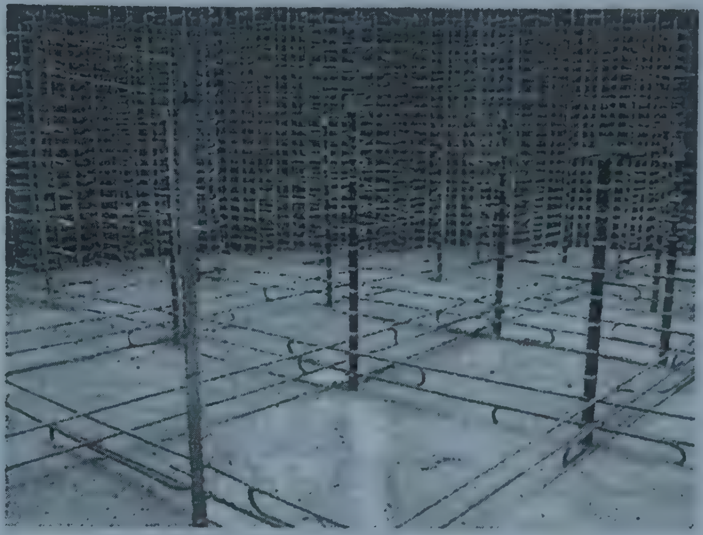
Temporary works for rebar arrangement shall be executed according to the work plan considering quality of arrangement works and safety in work execution. Bucking of the temporary frame for rebar arrangement works shall be carefully checked.

Rebar shall be fabricated according to the design document and specifications following relative regulation and codes. It shall also be placed and arranged at the location which is indicated at the design document.

Details of cover, gap, bending radius, anchorage, and connection which are demonstrated in design document shall be confirmed and shall be secured in the rebar arrangement works. Rebars shall be checked by their type, diameter, length, and quantity when delivery. Prevention of corrosion and stain shall be executed in storage in order to maintain the quality of the rebar. Fabrication of the rebars shall be done with the bending radii which are indicate in the design document, or in shop drawings. Cover, anchorage length, connecting method, placing method by tie wires, and safety of temporary frame shall be confirmed in actual arrangement or rebars at site.



Picture IV.5.1 Temporary Frame and placing by tie wires



Picture IV.5.2 Example of Temporary Frame of Intermediate Ties

6. Casting concrete

It is important to study the location and height of the scaffolding in order to secure the quality and filling easiness of cement concrete. The height shall not be too high that the cement and aggregate separate when casting and the condition of the cement concrete cannot be observed by direct sight. The height of scaffolding shall be set by the height difference of surface of the concrete which is already casted previously and the surface of scaffolding.

If a rod of the casting concrete for deep foundation is too large, height would be very high so that the securing the safety in rebar arrangement and temporary frame would be very difficult. Therefore, quantity of cement concrete for a rod shall be set considering the height of the deep foundation which will be casted with a single rod. It shall basically be set lower than 10m for a rod.

Separation of cement and aggregate shall be prevented in any cases, and it shall be checked by direct site. The joint of the concrete casting shall be treated so that it shall not be a defect of the structure.

Picture IV.5.1 Example of Compaction Work on the Buried Scaffolding

[Source: Handbook of Deep Foundation Construction, Japan Road Association]

(12) There is a retaining wall to hold the slope behind in constructing deep foundation on the steep slope which is called "Bamboo Split Shaped Retaining Method". Methodology and structural details of "Bamboo Split Shaped Retaining Method" are extracted from Design and Construction Manual of "Bamboo Split Shaped Retaining Method" of NEXCO Japan.

Extracted from Design and Construction Manual of "Bamboo Split Shaped Retaining Method", NEXCO

Chapter 1 General

1.1 Scope

This manual is applied for the planning, design, and construction of bamboo split shaped retaining method in order to excavate in circular cross section on the steep slope for the bridges on expressways of NEXCO.

1.2 Outline of the Method

This method constructs the retaining wall on steep slope by excavating the slope in bamboo split shape. It also stabilizes the cutting slope by inserting numbers of rebars/rock bolts to bedrock as reinforcing members.

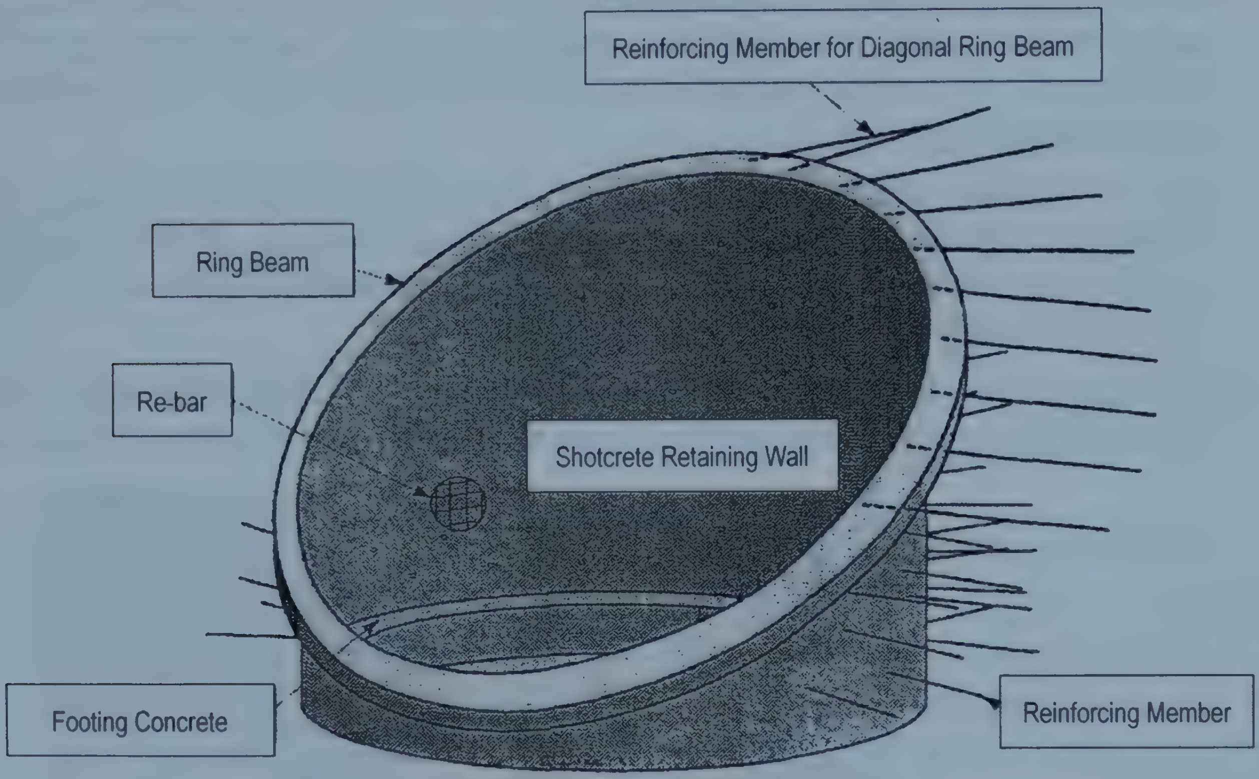


Figure 1-1 Image of "Bamboo Split Shaped Retaining Method"

Chapter 2 Planning

2.1 General

Safety, economic efficiency, workability, landscape, durability, and etc. shall be considered carefully in planning this method.

Flow chart of planning this method is shown in Figure 2-1.

This method can modify flexibly the arrangement and length of reinforcement members according to site condition revealed during construction because it is composed of simple mass production products.

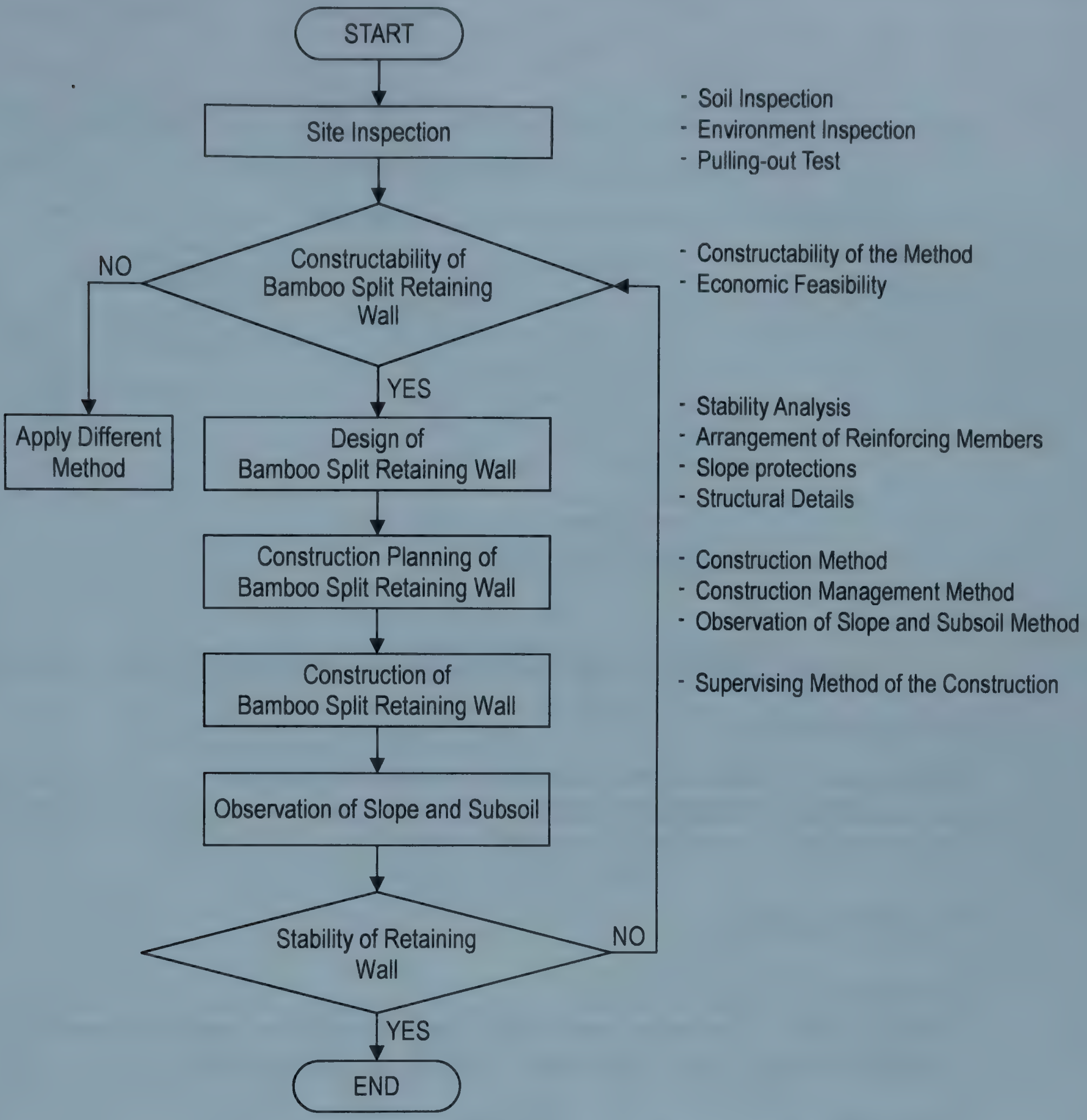

START
Site Inspection
- Soil Inspection
- Environment Inspection
- Pulling-out Test
Constructability of Bamboo Split Retaining Wall
NO
YES
- Constructability of the Method
- Economic Feasibility
Apply Different Method
Design of Bamboo Split Retaining Wall
- Stability Analysis
- Arrangement of Reinforcing Members
- Slope protections
- Structural Details
Construction Planning of Bamboo Split Retaining Wall
- Construction Method
- Construction Management Method
- Observation of Slope and Subsoil Method
Construction of Bamboo Split Retaining Wall
- Supervising Method of the Construction
Observation of Slope and Subsoil
Stability of Retaining Wall
NO
YES
END

Figure 1-1 Flow Chart of Planning Bamboo Split Retaining Wall Method

2.2 Scope of Method

This method is applied to bamboo split shaped excavation on steel slope of which the maximum excavation height is 5 to 20m and maximum diameter is 18m. If the maximum excavation height or diameter exceed 20m and 18m respectively, careful site inspection and detailed analysis, such as FEM analysis, shall be executed.

(1) Form of Structure

This manual is applied to the structure closed in-plane with ring beam.

(2) Excavation Height

This manual is applied to the excavation which has its maximum height of 5m and more. The maximum height of excavation could be up to 20m with less than 18m of excavation diameter. If the excavation height exceed this limit, there shall be a detailed study of reinforcement members and its restraint effect by FEM analysis, or with equivalent analysis method.

(3) Excavation Diameter

Excavation diameter shall be set with sufficient space for installing scaffolding for pier column construction, or other temporary equipment necessary for construction of pier column. (See Figure 2-2)

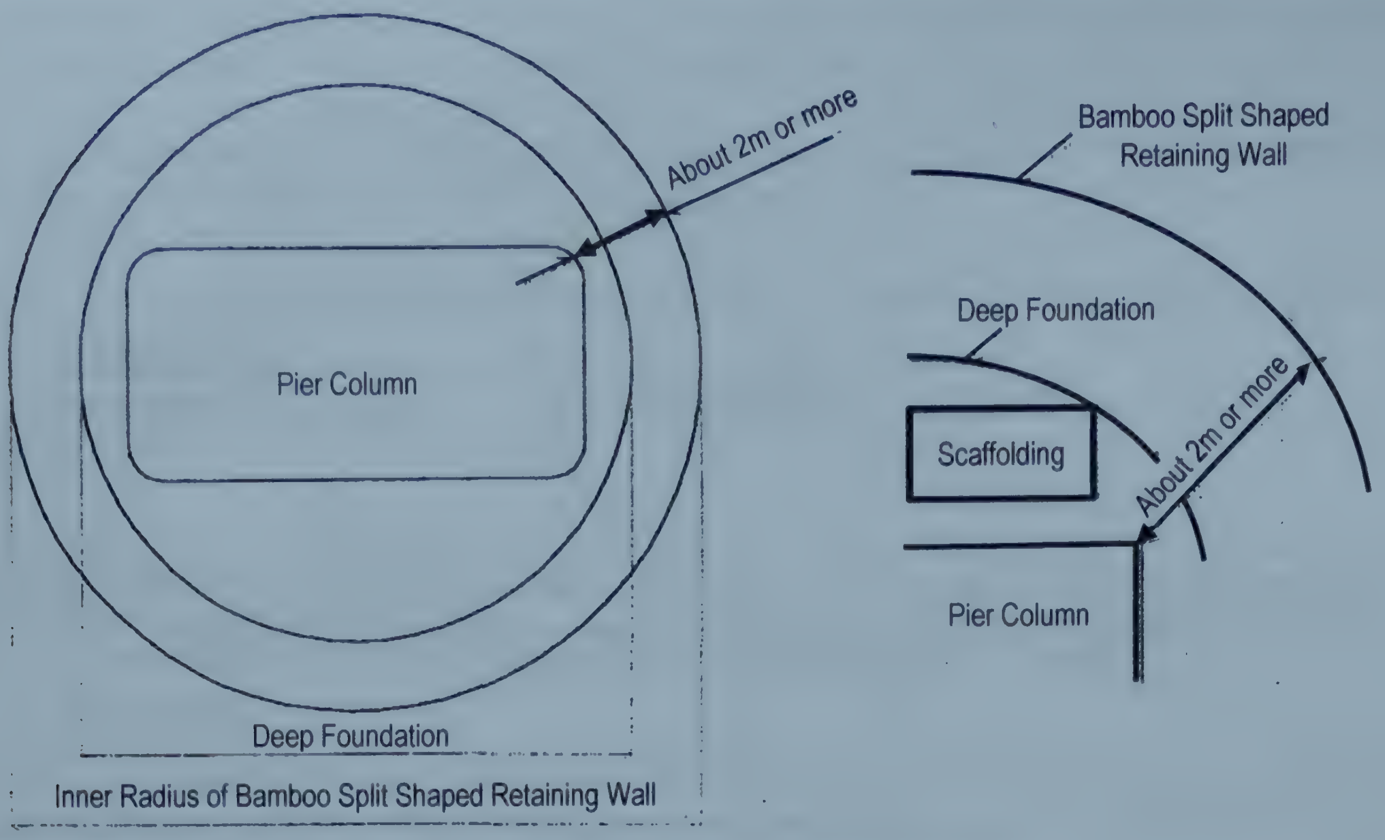


Figure 2-2 Example of Setting Excavation Diameter of Bamboo Split Shaped Retaining Wall

(4) Applied Subsoil

Applied subsoil for bamboo split shaped retaining wall is sandy soil (sand, gravel), soft rock (weathered rock, mud stone), and hard rock (fissured rock) which are stabilized when excavated by standard slope angle, or size of slope failure is estimated as very small.

Chapter 3 Inspection

3.1 General

Subsoil condition, underground water condition, and environmental condition of surrounding area shall be carefully inspected in order to plan, design, and construct with this method.

3.2 Inspection

Subsoil Inspection are executed in order to find out the structure of the subsoil and its engineering characteristics, underwater condition. It shall be executed applying appropriate inspection method and equipment.

There are basically 3 purposes of subsoil inspection. First is to judge the application of this method, second is for design, and third is for estimating the displacement during construction. Appropriate method and equipment shall be selected and executed to secure the efficiency and safety.

3.3 Inspection of Environmental Condition of Surrounding Area

Environmental condition of surrounding area shall be inspected considering the negative impact which execution of this method can raise. The result shall be referred to mitigate the impact and countermeasure shall be planned and executed following the environmental regulation of the area

3.4 Pulling-out Test

Pulling-out test shall be executed prior to design in order to find out the friction resistance value between the subsoil and reinforcement members, such as rebars, rock bolt. It shall be executed again before starting the construction to confirm the design friction resistance value.

Chapter 4 Design

4.1 General

Appropriate design method shall be applied according to each purpose of the construction stages.

(1) Structural details of bamboo split shaped retaining wall is as follows,

Table 4-1 Structural Details of Bamboo Split Shaped Retaining Wall

Excavation Height	5m to 20m (maximum height)
Ring Beam	Cross section: width 1.0m, height 0.6m
Reinforcement Member	- Area inside of arc of circular slip 1/2 of maximum height of each zone - Length and interval of reinforcement members Based on the calculation using the area mentioned above - Minimum length of reinforcing members Zone 1 Rock (grade CL): 3m Sand (including rock of grade D): 4m Zone 2 Rock, sand: 3m - Maximum interval of reinforcing members Minimum density of insert: 1/1.6m^2 Standard vertical interval: 1.2m
Shotcrete Wall	Thickness: 200mm (excluding 50mm thick primary shotcreting) Rebar arrangement: minimum requirement of rebar
Footing Concrete	Thickness: 500mm Rebar arrangement: D16@300 D16@150 near the edge

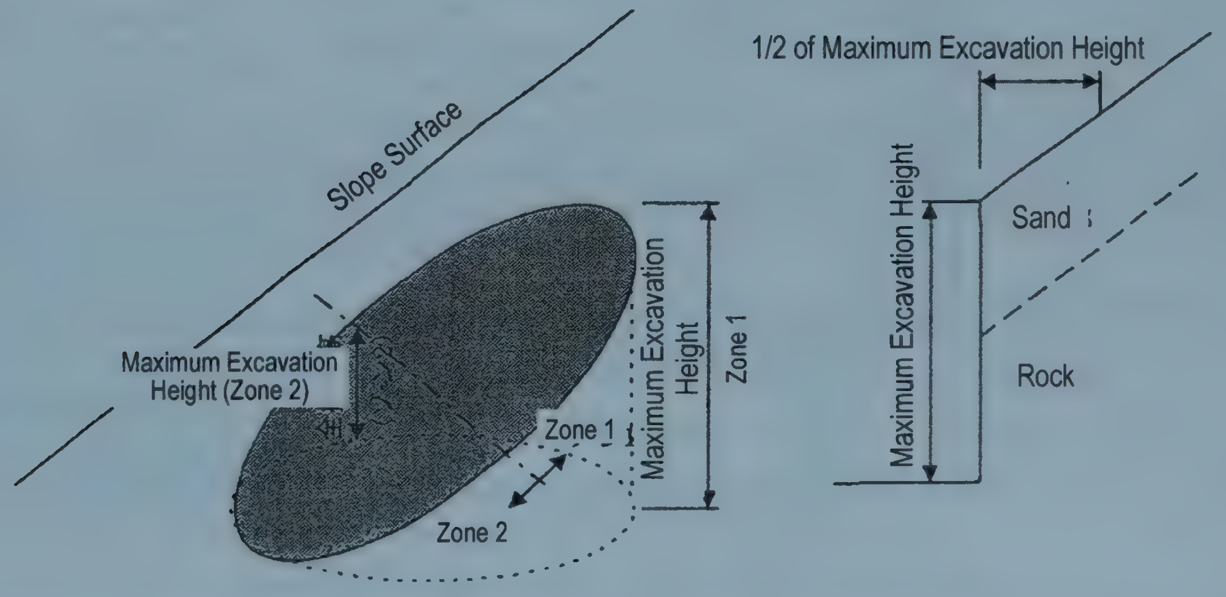

Slope Surface
Maximum Excavation Height (Zone 2)
Zone 1
Zone 2
Maximum Excavation Height
Zone 1
1/2 of Maximum Excavation Height
Maximum Excavation Height
Sand
Rock

Figure 4-1 Area of Subsoil where Bamboo Split Shaped Retaining Wall Method is Applied

(2) Flow chart of design of this method is shown in Figure 4-2.

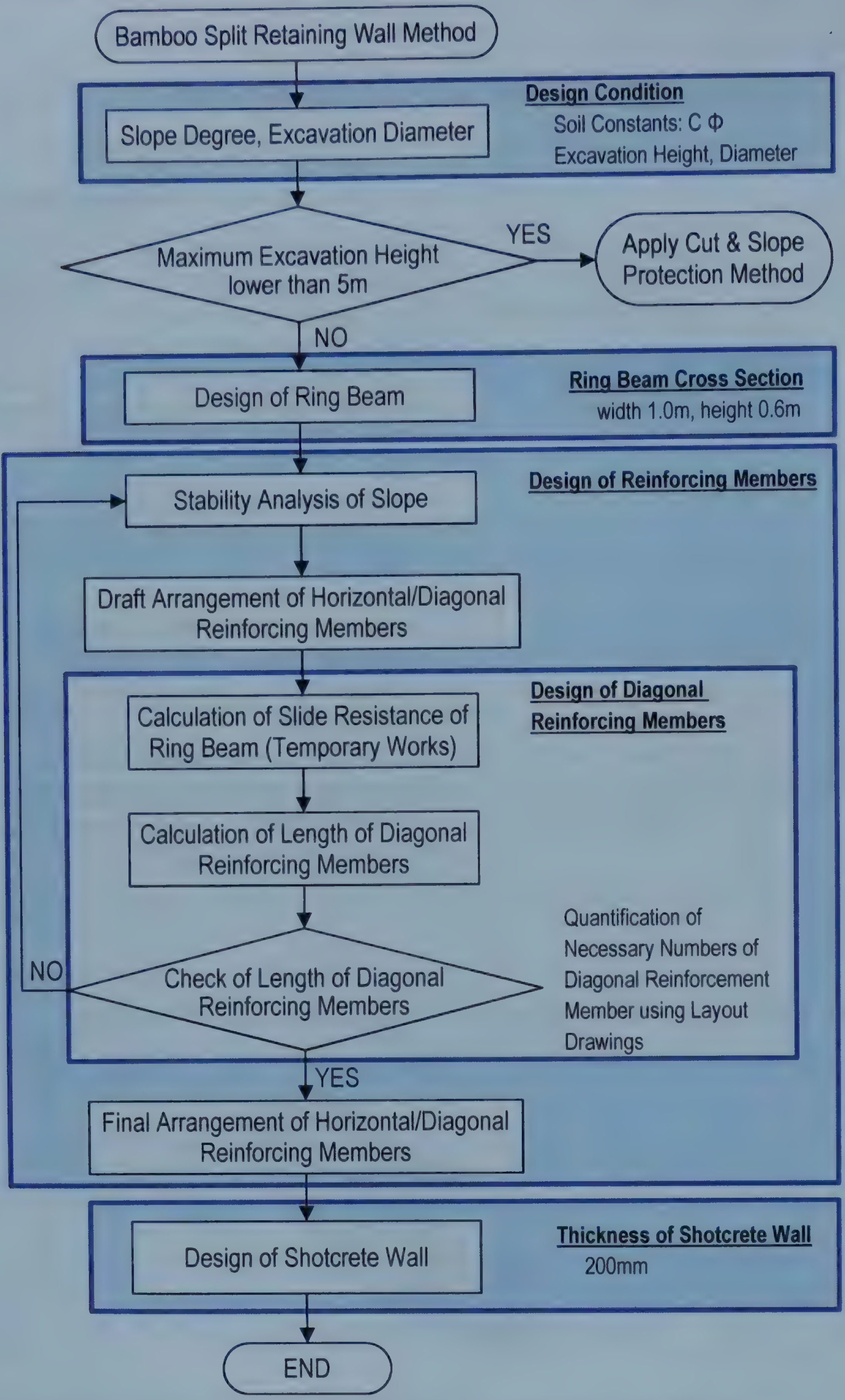

Bamboo Split Retaining Wall Method
Slope Degree, Excavation Diameter
Design Condition
Soil Constants: C Φ
Excavation Height, Diameter
Maximum Excavation Height lower than 5m
YES
Apply Cut & Slope Protection Method
NO
Design of Ring Beam
Ring Beam Cross Section
width 1.0m, height 0.6m
Design of Reinforcing Members
Stability Analysis of Slope
Draft Arrangement of Horizontal/Diagonal Reinforcing Members
Design of Diagonal Reinforcing Members
Calculation of Slide Resistance of Ring Beam (Temporary Works)
Calculation of Length of Diagonal Reinforcing Members
Quantification of Necessary Numbers of Diagonal Reinforcement Member using Layout Drawings
NO
Check of Length of Diagonal Reinforcing Members
YES
Final Arrangement of Horizontal/Diagonal Reinforcing Members
Design of Shotcrete Wall
Thickness of Shotcrete Wall
200mm
END

Figure 4-2 Flow Chart of Deisgn of Bamboo Split Retaining Wall Method

4.2 Soil Constants

Soil constants for designing this method, such as unit weight γ, cohesion c, and internal friction angle Φ, shall be set using the result of subsoil inspection. It can be calculated from (1) collapse of the slope nearby, (2) excavation condition, and (3) laboratory test.

4.3 Study of Internal/external stability

Respective safety factor shall be secured for both internal and external stability. Internal stability shall be checked by circular slip surface method, and external stability shall be checked by slide of reinforcing area.

4.4 Safety Factor and Allowable Stress

(1) Design Safety Factor of Internal Stability

Design safety factor of internal stability shall be set respectively for temporary works (during construction) and after completion (as a permanent structure).

Table 4-2 Design Safety Factor of Reinforced Slope

Item	Design Safety Factor
Permanent Slope	1.2
Temporary Slope (during construction)	1.05

Design safety factor for temporary slope is applied from the beginning of the excavation to installation of the last reinforcement member.

(2) Design Safety Factor of External Stability

Design safety factor of internal stability is shown in the table below.

Table 4-4 Design Safety Factor of Reinforced Slope

Design Safety Factor	1.5

(3) Allowable Stress of Reinforcing Members

Allowable stress of reinforcing member is set that dividing allowable stress of rebar by 1.7.

Table 4-5 Allowable Stress of Rebars

Type of Rebar	SD345
Allowable stress	200 (N/mm^2)

(4) Ultimate Surface Friction Resistance between Injection Material and Subsoil

Ultimate surface friction resistance between injection material and subsoil is set based on the results of pulling-out test. However, if the pulling-out test prior to design is not available, estimated value can be utilized for design in condition of confirming the value before commencement of actual excavation works.

Table 4-6 Safety Factor of Ultimate Surface Friction Resistance

Item	Design Safety Factor
Permanent Slope	2.0

(5) Allowable Adhesive Stress between Reinforcing Member and Injecting Material

Allowable adhesive stress between reinforcing member and injecting material for permanent structure is shown in the table below, based on the description of Specifications for Highway Bridges of Japan (Part III Concrete Bridge)

Table 4-7 Allowable Adhesive Stress between Rebar and Injecting Material (N/mm^2)

Design Compressive Strength of Injecting Material	24	27	30
Allowable Adhesive Stress	1.6	1.7	1.8

(6) Safety Factor of Slide for Ring Beam

Safety factor of slide for ring beam is shown in the table below.

Table 4-8 Design Safety Factor of Slide for Ring Beam

Design Safety Factor	1.20

(7) Allowable Stress of Shotcrete (Standard Design Strength: 18 N/mm^2)

Allowable stress of shotcrete is shown in the table below.

Table 4-9 Allowable Stress of Shotcrete (N/mm^2)

Allowable Bending Compression Stress	6.0
Allowable Shear Stress	0.21
Allowable Adhesive Stress	1.2

(8) Allowable Stress of Rebar

Allowable stress of rebars for ring beam, shotcrete wall, and footing concrete is shown in the table below

Table 4-10 Allowable Tensile Stress of Rebar (N/mm^2)

Allowable Tensile Stress of Rebar	180

4.5 Specification of Material

(1) Cement Concrete

Design standard strength of cement concrete is shown in the table below.

Table 4-11 Design Standard Strength of Cement Concrete

Item	Design Standard Strength
Shotcrete	18
Footing Concrete	24

(2) Steel

Following standard shall be satisfied for the steel materials.

Table 4-12 Standard of Steel Materials

Type of Member	Standard	Remarks
Rebar	SD345	
Reinforcing Mesh	JIS G 3551	Φ5 x 150 x 150mm
Reinforcing Member	SD345	D19~D25
Bearing Plate	SS400	PL-150 x 150 x 6mm

(3) Injecting Materials

Design standard strength of injecting material is more than (or equal to) 24 N/mm^2. Injecting material is basically products made of cement milk. It shall have to have sufficient strength, long term stability, and fluidity.

(4) Corrosion Prevention

Corrosion prevention for reinforcing members, bearing plates and nuts shall be done by galvanizing. Corrosion allowance of 1.0mm shall be considered for reinforcing member. Therefore, design diameter of the reinforcing members is 1.0mm less than nominal diameter of the members.

Table 4-13 Standard of Corrosion Prevention

Type of Member	Standard
Reinforcing Member, Bering Plate, Nut	JIS H8641 Type 2, HDZ55

4.6 Design of Ring Beam

Ring beam forms rigid circular frame on the ground surface prior to excavation in bamboo split shape. It is installed in order to regulate the displacement of upper section of shotcrete wall.

Specifications of the members for ring beam are shown in the table below.

Table 4-15 Specifications of the Members for Ring Beam

Item	Value
Cross Section of Ring Beam	width 1.0m, height 0.6m
Design Standard Strength	18 N/mm^2
Design Allowable Displacement	6mm (one side)

4.7 Diagonal Reinforcing Member for Ring Beam

Diagonal reinforcing member for ring beam is installed in order to prevent sliding in installing ring beam and as a primary reinforcing member.

Construction area, interval of installation, and resistance against slide is shown in the table below.

Table 4-16 Specifications of Diagonal Reinforcing Member

Item	Value/Description
Construction Area	180 degrees towards upper side of the slope
Interval of Installation	1000 to 1300mm
Resistant Effect against Slide	Pulling and Fastening

4.8 Design of Shotcrete Wall

Shotcrete wall is installed in order to resist against the soil pressure from the back side and protect the slope.

Specifications of the members for shotcrete wall are shown in the table below.

Table 4-16 Specifications of the Members for Shotcrete Wall

Item	Value/Description
Thickness of Wall	200mm (excluding 50mm thick primary shotcreting)
Rebar	D16@300 (vertical and horizontal)
Reinforcing Mesh	Welded Mesh Φ5 x 150 x 150mm (primary shotcreting)

4.9 Design of Reinforcing Members

Reinforcing members are installed in order to improve the stability of the slope by integrating the retaining wall with the slope, as well as regulating the displacement of the retaining wall. Cross sections of the analysis are set respectively for Zone 1 (upper half of the retaining wall), and Zone 2 (lower half of the retaining wall). Analysis is done at the cross section with maximum excavation depth by slice dividing method and ultimate balanced stability method. Depth of the analysis is 1/2 of the maximum excavation depth.

Minimum length of reinforcing member in Zone 1 is 4m (sand), 3m (rock), and Zone is 3m (sand and rock)

4.10 Arrangement of Reinforcing Members

Reinforcing members shall be arranged to realize the designed reinforcing effect to the retaining wall with appropriate intervals. Details of arrangement is based on the arrangement of reinforcing members of plane slope protection.

Reinforcing arrangement shall be decided based on the result of boring inspections one in mountain side and another is on valley side of the excavation location. Boring log and development plan of reinforcing members shall be provided for the decision of arrangement.

Specification of arrangement of reinforcing member is shown in the table below.

Table 4-16 Specification of Arrangement of Reinforcing Member

Item	Value/Description
Interval of Reinforcing Members	Vertical: 1.2m, Horizontal: 1.0-1.3m
Minimum Density of Insert	1/l.6m^2
Insert Angle of Reinforcing Members	10 degrees downward from level
Maximum Interval	Vertical: 1.2m, Horizontal: 1.3m

4.11 Design of Footing Concrete

Footing concrete is installed in order to sustain the shape of bottom surface of excavated hole (regulate the displacement).

Specifications of footing concrete are shown in the table below.

Table 4-16 Specifications of the Members for Shotcrete Wall

Item	Value/Description
Thickness of Footing	500mm
Rebar	D16@300, D16@150 near the edge
Reinforcing Mesh	More than 40 times of diameter

Chapter 5 Construction

5.1 Construction Planning

Construction planning shall be executed after careful review of design condition, topographic and geotechnical condition, and site condition, and planning the detailed procedure securing safety and preserving natural environment of surrounding area.

5.2 Construction Machinery and Yard

Site condition shall be carefully reviewed to plan construction machinery and yard.

5.3 Construction Procedure

There are three main work items, such as installing ring beam, shotcrete wall, and footing concrete. Construction procedure shall be carefully selected considering the site condition.

(1) Flowchart of Construction of this method is shown in Figure 5-1.

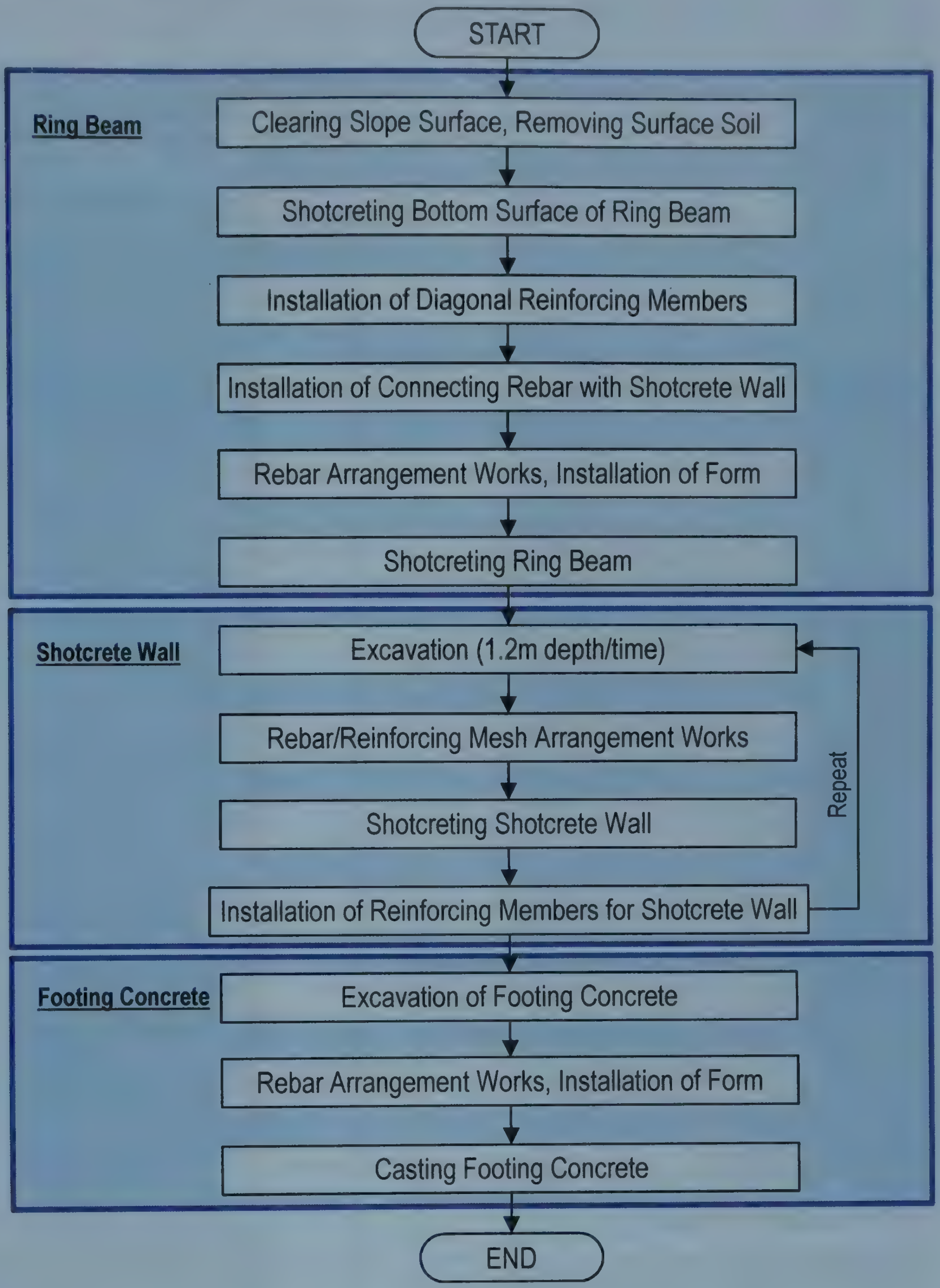

START
Ring Beam
Clearing Slope Surface, Removing Surface Soil
Shotcreting Bottom Surface of Ring Beam
Installation of Diagonal Reinforcing Members
Installation of Connecting Rebar with Shotcrete Wall
Rebar Arrangement Works, Installation of Form
Shotcreting Ring Beam
Shotcrete Wall
Excavation (1.2m depth/time)
Rebar/Reinforcing Mesh Arrangement Works
Shotcreting Shotcrete Wall
Installation of Reinforcing Members for Shotcrete Wall
Repeat
Footing Concrete
Excavation of Footing Concrete
Rebar Arrangement Works, Installation of Form
Casting Footing Concrete
END

Figure 4-2 Flow Chart of Construction of Bamboo Split Retaining Wall Method

5.4 Construction Management

Management items shall be decided based on the design documents and carefully executed to secure the quality in construction management.

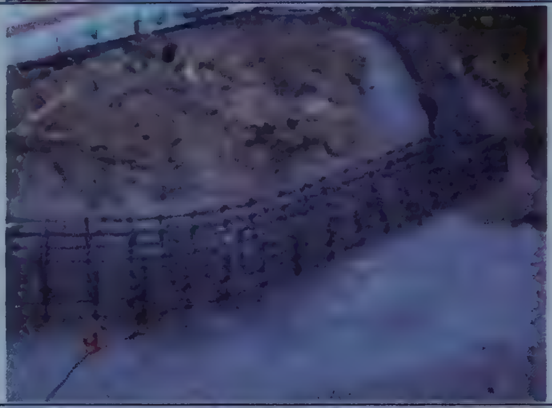
5.5 Observation of Slope and Subsoil during Construction

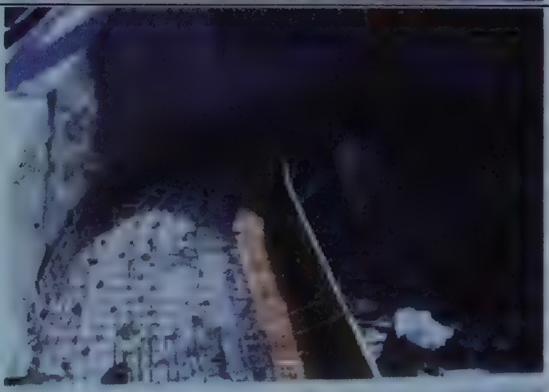
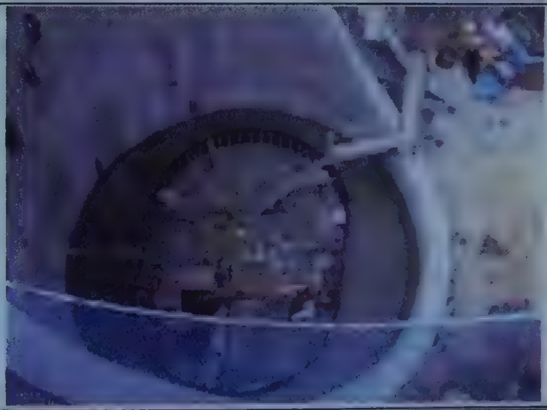
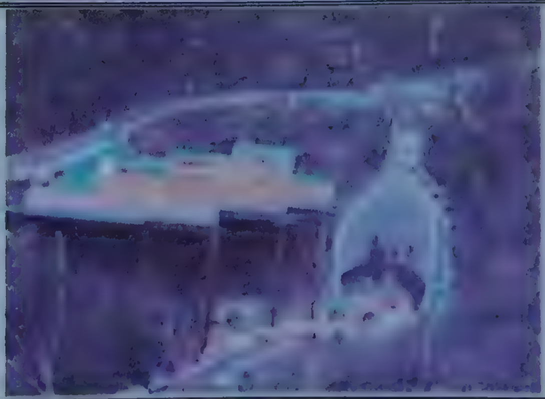
This method shall be constructed observing the slope and subsoil throughout the construction stage.
Observation period starts before the first excavation stars, and ends after the completion of the entire works of deep foundation.
Frequency of the observation shall be once a day. It is obligatory to observe the day of excavation.

[Source: Design and Construction Manual of “Bamboo Split Shaped Retaining Method”, NEXCO]

(13) Actual images of construction procedure of Bamboo Split Shaped Retaining Wall Method is demonstrated in the table below.

Table 4-1: Actual images of construction procedure of Bamboo Split Shaped Retaining Wall Method

Construction Procedure		Image
Ring Beam	Clearing Slope Surface	
	Shotcreting Bottom Surface of Ring Beam	
	Rebar Arrangement Works, Installation of Form of Ring Beam	
	Shotcreting Ring Beam	
Shotcrete Wall	Excavation of Rock	

	Repetition of Excavation near the Shotcrete Wall and Shotcreting	
Footing Concrete	Excavation of Footing Concrete	
	Rebar Arrangement Works, Installation of Form	
	Casting Footing Concrete	
Completion		

4.3.5 Caisson Foundation

(1) Caisson foundation can be applied at the bottom of the valley where the bearing layer of subsoil is deep and subsoil above the bearing layer consists of large gravels. Bored piles are difficult to be constructed with this type of subsoil conditions.

(2) There are two types of caisson foundation, one is open caisson and another is pneumatic caisson. Open caisson is applied where underground water level is low, while pneumatic caisson can be applied with high underground water level.

(3) Pneumatic caisson gives pressure inside the caisson and hold the underground water outside of the caisson. There would be very high air pressure inside of caisson and would be very difficult environment for labour. Therefore, automation of construction of pneumatic caisson has been developed to minimize the labour to go inside the caisson while high pressure is given.

(4) There is a chapter specified for construction of caisson foundation in Specification for Highway Bridges in Japan (Part IV Substructure). The contents are quoted as below.

Quoted from Specifications for Highway Bridges of Japan (Part IV Substructures)

CHAPTER 17 Construction of Caisson Foundation

17.1 Method to be Applied

> This chapter shall be applied to the caisson foundations constructed by the open caisson method or pneumatic caisson method.

The caisson foundation dealt with in this chapter means one for which an open-end cylindrical structure made of concrete is produced on the ground and is gradually sunk underground by eliminating earth out of the cylinder interior, until it reaches a specified bearing layer.

The caisson foundations can be roughly classified into an open caisson and a pneumatic caisson. Here, the open caisson method is a construction method in which the caisson is sunk to reach the predetermined bearing layer while the earth in the caisson is excavated and removed by a clam shell, grub bucket or the like. The pneumatic caisson method is a construction method in which the caisson is sunk to reach the predetermined bearing layer while a working chamber is established beneath the caisson, compressed air is fed to the working chamber to remove the water in it, and the earth there is excavated and removed manually or by machinery.

In the caisson construction method, improvements in construction techniques such as unmanned and automated are advanced, and also safety has been improved, it has become possible to increase the scale and depth. Where such improved methods are adopted, full-understanding of its contents and, if necessary, inspection on items other than those stated in this chapter will be required.

17.2 Selection of Construction Machines and Instruments

> The construction machines and instruments shall be chosen taking into account of the dimensions of the caisson, environment of the work site, condition of the ground, safety of work, and the like.

Between the open caisson ·and pneumatic caisson methods, major construction machines and instruments are essentially different because of the difference in their construction process.

"Construction machines and tools for the excavating process" refers mainly to equipment for underwater excavation in the open caisson method and temporary facilities (electric power equipment, air-supply systems, excavation equipment, soil removal equipment, and safety equipment) and rigging equipment (locks, shafts, air-supply pipes, exhaust pipes, etc.) in the pneumatic caisson method. As a consequence of planning a sinking process under a theoretical sinking diagram of a caisson foundation, facilities used by the two methods differ depending on whether applied loads and friction-reducing mechanisms are necessary.

When choosing major machines and instruments for the construction of a caisson foundation, the type, capacity, quantity and the like of the machine should be fully examined in view of the various conditions concerning the construction. Especially when it is estimated from a theoretical sinking diagram that forced sinking with loading or friction-reducing mechanism is needed, their methods, materials, and timings should be taken into account. It is also necessary in the pneumatic caisson method that scales of air-delivery facilities and other related ones may be changed depending on types of soil layers and sinking methods.

In terms of the details of instruments of pneumatic caisson method, "Construction Manual for Pneumatic Caisson" (Japan Pneumatic Technology Association, September, 2002) and other documents may be referred.

17.3 Cutting Edges

The cutting edge of a caisson shall be constructed to ensure that the caisson can be immersed safely.

The cutting edge portion of a caisson is the first portion to penetrate the ground when the caisson is immersed, and there are complex external forces acting here compared to the side walls. The shape of the cutting edge has a large influence on the sinking behavior of the caisson. Therefore, the shape of the cutting edge and cutting edge hardware should take a structure that duly considers the soil conditions. By the way, attaching a flat bar to the cutting edge is effective in reducing the skin friction force and reinforcing the cutting edge point.

17.4 Installation of a Caisson

A caisson shall be installed at soil layers which fully bear weights of the caisson main body, forms, and center materials, and which assure safe initial sinking.

A caisson is installed on the ground, on a bank, or in water. The most appropriate method should be chosen by considering construction methods, water depths, and weight of concrete.

Unlike the pneumatic caisson, the uplift pressure due to the working air pressure cannot be expected of the open caisson once caisson immersion is started. Therefore, steep sinking or declining often occurs. For these matters, it is necessary to take preventive measures beforehand as described in Clause 17 .6. If an underwater installation method is to be employed, firm guide stoles and suspension facilities are required for attitude control during installation and initial sinking of the caisson.

17.5 Caisson Main Body

A caisson main body shall be constructed to satisfy its functional requirements by taking construction conditions into account.

1) A construction working chamber should have a one-piece structure which assures air and water tightness. Therefore, concrete should be continuously placed in principle.
2) General concrete structures are relatively rarely moved after concrete placement. In construction by the caisson method, a long curing period sometimes cannot be taken due to the characteristics of the excavation and sinking work performed after concrete placement, or from the perspective of construction efficiency. Because complex loads involved in these operations will have to be borne by an early-aged caisson body in such a case, a full examination is necessary when setting the curing period.
3) Because slanting and displacement are apt to occur in an early stage of caisson sinking, it is good to plan the initial construction height considering the dimensions of the caisson, properties of the ground and the like, and for the subsequent construction heights as well, decide them considering the sinking diagram prepared during construction planning, the construction situation and the like.
4) The caisson should not be immersed with long reinforcement in an unsupported upright position or with the concreting forms in the portion not concreted because of a change of concreting height attached to it.

5) Concrete joints must be watertight to resist water loads while sinking a caisson and to meet requirements after construction. So, carefully treat the joints by removing laitance and chipping them.

6) Because even a slight eccentric load can cause slanting depending on the ground conditions and the like, strict construction control is required on the loads due to temporarily placed materials and surplus soil.

7) During constructing a caisson, specifically during casting concrete, it may become more difficult to keep the balance of whole system due to the increase of the body weight. Therefore, considering work safety, excavation and immersion must be suspended during caisson construction to monitor the settlement due to the added weight.

17.6 Excavation and Sinking

(1) Excavation shall be carried out while modifying a sinking diagram if so required by construction and soil layer conditions, and abrupt caisson sinking shall be avoided while care is taken of inclination, movement, and rotation of the caisson.

(2) A caisson shall be sunk, in principle, by using its weight, applied loads, and reduction of frictional resistance with careful attention for not disturbing surrounding ground and bearing ground.

(1) Since a sinking diagram is prepared at a planning stage by including some assumptions such as skin frictional resistance and cutting edge resistance, construction should be safely conducted by correcting these conditions, if necessary, during the excavation and sinking processes.

It is important in these processes to protect a caisson from moving, tilting, or abruptly sinking. Since these phenomena tend to happen as a result of collapse of the ground, inclination of soil layers, and eccentric earth pressure, attention should be paid to these factors. It is necessary to correct any movements or inclination of the caisson at a relatively shallow embedment (1 to 2 lifts), because their correction usually becomes rather difficult when it is embedded more than two times the shorter side of the caisson.

1) For the pneumatic caisson, earth should not be dug beneath the bottom plane of the cutting edge as a general rule. In some grounds, immersion may be difficult without fu1iher excavation. Note, however, that the "Ordinance on Safety and Health of Work under High Pressure" prohibits digging down 0.5 meter or more.

A pneumatic caisson may sometimes cause sudden sinking due to small skin frictional resistance and cutting edge bearing resistance at a stage of relatively shallow embedment (1 to 2 lifts) or at soil layers whose intermediate layer is soft. Hence, the sinking procedures should be adjusted by shortening a lift length of the caisson or providing a saddle in the working chamber interior, based on the sinking diagram. Ground excavation in such a case should be conducted carefully by human labor: mechanical excavation using a bulldozer or a bucket should be avoided.

In deep embedment into hard ground, such as soft rock and a hardpan layer, if the load exceeds the sinking resistance force, rapid sinking may occur, generating vibrations and affecting the surrounding environment and the safety of construction. Accordingly, it is necessary to take measures to prevent rapid sinking, such as doing sinking control with a saddle or the like constructed in the working chamber, and replacement of earth and sand surrounding the cutting edge.

2) Unlike a pneumatic caisson method, it is impossible in the open caisson method to correct inclination of a caisson at its bottom without difficulty. Hence, causes of inclination by external forces must be minimized. It is desirable to avoid sinking a caisson at banks and such other locations as are susceptible to eccentric earth pressure. Should it be inevitable to sink it at such a location, measures should be taken to prevent eccentric pressure, and dynamic relationship of caisson sinking should be reviewed in detail beforehand.

In case of the wet excavation, a ponding level in a caisson should be kept as high as a groundwater one. This is effective in preventing rupture in bottom layers which have been disturbed by the excavation or excavated bottom surface to be caused by excessive excavation. Care must be taken against occurrence of boiling or heaving phenomena which are caused by differences in water levels between inside and outside the caisson, because they may develop sudden settlement of the caisson, loosening of ground around the caisson external periphery, and reduction in bearing capacity of the bottom soil layers.

Generally, at an initial sinking of a caisson, the caisson is unstable against inclination and movement, and no uplift can be expected unlike the case with the pneumatic caisson method, thereby sudden settlement often occurring as excavation proceeds. Hence, excessive excavation should be avoided below a cutting edge as far as possible; excavation should be kept within a range necessary for sinking the caisson. If sinking becomes difficult again, some loads such as ingots may be placed on the .caisson upper side to continue the sinking operation.

(2) 1) For the pneumatic caisson, immersion generally takes place by pouring water in the hollow part of the caisson body when sinking by its own weight alone becomes difficult. The amount of water in this case shall be controlled based on the sinking diagram.

To sink the pneumatic foundation to be bearing ground in a safe and sure way, there are methods of reducing the friction force between the ground and caisson wall surface. The most popular of the friction reduction methods is a friction cut provided at the cutting edge of the caisson. Its dimensions are generally about 50 mm, though they depend on the ground. Besides this, the methods of lubricant injection and the like are available as a method for reducing friction force.

2) For the pneumatic caisson, contact grouting may have to be done as a general rule, depending on the friction cut dimensions and ground conditions. This work is to inject a cement paste, cement bentonite or other filler having a strength equivalent or superior to the ground into the gap between the ground and caisson wall surface after completion of immersion to recover the strength of the peripheral ground and secure the adhesion between the caisson and peripheral ground. The mixing of the filler must be decided considering the ground conditions, constructability and the like. Following past results, the strength is taken at 3 N/mm^2 in most cases. If there is hard rock or the like around the caisson and the rock has higher strength than the concrete of the caisson body, the strength of the filler may be taken the same or more than the concrete strength of caisson body. This is because the filler strength is taken at about the maximum unit horizontal ground reaction, it is possible to secure the adhesion between the caisson and peripheral ground without letting the filler be broken. When doing contact grouting, the influence on the water quality of the neighborhood should be considered.

3) In the construction of a pneumatic caisson, the work air pressure shall be basically set to a pressure in agreement with the pore pressure in the ground at the edge of the blades. However, if the work air pressure exceeds the pore pressure in the ground at the edge of the blades for some reason, air leakage of compressed air into the surrounding ground will occur (air blow). This phenomenon may cause looseness of the surrounding ground due to the outflow of fine-grained particles, blowout into nearby wells or basements, and generation of oxygen deficient air due to oxidation of air leaking into some types of soil. Accordingly, it is necessary to make sufficient considerations for air leakage prevention in order to take appropriate preventive measures. Preventive measures against air leakage include underwater excavation performed with the water level higher than the cutting edge position, use of an automatic electrical pressure adjuster, construction of an observation well, and measurement management. An appropriate combination of these measures must be selected with consideration given to on-site conditions. In addition, if a boring exploration borehole is not properly restored to its original condition, sinking of the caisson may lead to failure such as blowout of ground water from the borehole and leakage of compressed air, disturbing construction. Therefore, boring exploration boreholes must be carefully restored to their original condition so that such failure will not occur.

4) When the sinking resistance against the open caisson becomes large, immersion takes place by imposing a sinking load, or injecting a lubricant or other sinking acceleration work to reduce the skin friction force.

Sinking load imposition or sinking acceleration work of the open caisson should be employed taking into account the sinking diagram examined beforehand and duly considering the soil conditions at the site.

For the open caisson, employing the press-in method as sinking acceleration work is effective if the sinking force is substantially insufficient. However, because the caisson body has small rigidity, employing the press-in method in an early stage of immersion may cause cracks in the caisson body due to press-in. Therefore, it is good to employ it after the caisson body rigidity has been secured adequately by constructing a second lift or second and subsequent lifts or otherwise.

At the excavation immediately before the final sinking of the open caisson, it is good to avoid overexcavation in the central area as far as possible. Like pneumatic caissons, fillers may be inserted into an opening above the friction cutter after the caisson is sunk, depending on construction and ground conditions.

17.7 Verification of Bearing Layer

The performance of a bearing layer shall be evaluated _from a comprehensive range of perspectives by various approaches, such as checking of the immersion depth of the caisson, the soil properties, and the bearing capacity and deformation characteristics of ground;

1) The pneumatic caisson is a construction method in which the conditions of the bearing layer can be directly ascertained. As a technique to ascertain the bearing capacity and deformation characteristics of the ground, the plate bearing test which generally uses the working chamber ceiling slab is available (See Reference 8). However, if an excessive amount of compressed air is applied to dry the ground in the test, air leakage may occur. Accordingly, as described in Reference Material 8, since the test is performed in the environment of the caisson working chamber, it is desirable to take measures for air leakage prevention and to perform a multi-cycle rapid loading test. Specifically, as a measure for air leakage prevention, the cutting edge is embedded sufficiently into the bearing layer to make the loading surface relatively high.

   On completion of the test, find the ultimate unit bearing capacity and the coefficient of vertical ground reaction and examine the strength and deformation characteristics of the ground. Moreover, bearing of the relevant caisson should be judged in a comprehensive manner considering the state of the soil layers.

   In general, when the caisson has reached near the bearing layer, it is desirable to execute the work carefully so as not to disturb the bottom ground. What is more, it is necessary to do the work while examining the ground condition beforehand by sounding to be ready for adjusting the caisson length.

2) When ascertaining the bearing capacity of an open caisson, the immersion of the caisson to the bearing layer assumed in the design is generally ascertained based on the soil profiles surveyed beforehand and the geological condition, soil layers, and excavated soil control at excavation. Therefore, if the ground at the completion of immersion is different from that at the design and there is anxiety about the bearing capacity judging from the excavated soil, it is necessary to do a boring survey at the caisson position to ascertain the bearing layer and bearing capacity. When the press-in method is employed, the bearing layer can be ascertained by the press-in force.

   When dewatering is possible and safe, the bearing capacity can be ascertained by doing a plate bearing test.

17.8 Top Slab

A top slab of a caisson shall be constructed in a manner to securely transfer applied loads to the caisson main body.

A top slab is an important member that transmits loads acting on bridge portions in positions higher than that of the slab to the ground via the side walls and partition wall of the caisson. Accordingly, it is necessary to construct the top slab body reliably and to connect the top slab reliably to the bridge body and the side walls and partition wall.

17.9 Cut-off Walls and Temporary Retaining Walls

> Cut-off walls and temporary retaining walls shall have specific water-stopping capability, have a structure capable of bearing earth pressure, hydraulic pressure, and other external forces during a caisson-sinking process, and can be removed after completion of work.

For the cut-off wall and temporary retaining wall, the materials and the like shown in Table-C. 17.9.1 are generally used considering the construction conditions, construction speed, economic efficiency and the like.

When dismantling and removing cut-off walls and temporary retaining walls, care must be taken not to damage pier bodies and caissons, and measures should be taken a keep external forces between inside and outside the cut-off wall by supplying water, filling it back, or excavating the ground.

Table-C.17.9.1 Comparison of Properties among Cut-off Walls and Temporary Retaining Walls

Items Materials	Recommended wall height	Water-stopping capability	Water stopping at joints	Workability	Easy to remove	Removing method	Remarks
Plain concrete	1.5 m or less	○	—	○	Δ	To break by using a rock drill, cutter, dynamic, or electric heater	To be used for cases which do not need high wall height and no tensile force is expected to occur in the cross section due to a caisson's shape.
Reinforced concrete	1 - 5 m	○	—	○	Δ		Can be easily constructed as an additional part of concrete construction of a caisson main body, but it is necessary to review measures to remove the walls in advance.
Steel sheet pile	3 - 8 m	Δ	Water-stopping agent, packings, and mortar	○	○	To unfasten tie rods and bolts and then to pull out	Tie rods and other fastening metals should be used not to allow the sheet piles to be separated from the caisson top end due to wall friction force against earth while sinking the caisson.
Steel panel and precast concrete block	2 - 5 m	Δ		Δ	○		

○ : Generally agreeable
Δ : Thorough review is needed before adopting
— : Generally inapplicable

17.10 Center

> (1) A center shall have a structure strong enough to bear all the loads to occur while installing a cutting edge and a working chamber ceiling slab.
>
> (2) Ground on which the center is built shall be able to safely bear all the weights of the center, working chamber, etc.
>
> (3) Dismantling of a centering shall be done ascertaining the concrete strength after placement and taking care not to impose an adverse effect on the structure.

(1) The structure of a centering should be examined considering the construction of the working chamber. According to the Ordinance on Industrial Safety and Health, the height of the working chamber is

specified to be 1.8 meters or more. However, because machinery excavation is generally employed, it is taken at 2.3 meters or more in most cases, considering the safety and the height of the excavation machinery.

Centers are classified into wooden, steel, and earth types according to their raw materials.

The whole center, wooden or steel, is easily influenced by distortion in many cases when non-uniform settlement or inclination occurs: it is necessary to arrange diagonal braces whenever required. During dismantlement, it is necessary to take appropriate measures, with consideration given to the new surface form of the caisson so that inclination will not occur due to eccentric loads. When an earth centering is to be constructed by banking on the site, the settlement of the banked earth may adversely affect structures in the concrete placement in the working chamber. Accordingly, careful examination is required before adoption of this approach.

(2) Soil layers on which a center is installed should be so much strong as to fully bear gravities of the center, cutting edges, and working chamber ceiling slabs. Should the soil layers run short of bearing capacity, measures to improve them are needed in advance not to cause non uniform settlement or inclination of a caisson. Concrete should be placed uniformly.

(3) When dismantling the centering, concrete strength must have developed to the extent that the dismantling will not exert an adverse effect on the structure. For the compressive strength of concrete when the centering may be dismantled, a strength development of 14 N/mm^2 or more may be taken as a rough standard.

17.11 Concrete Filling of a Pneumatic Caisson

A working chamber shall be certainly filled with concrete of workability suitable for the construction conditions while regulating air pressure in the working chamber and considering preventing from clogging of pipes, after correcting irregularities of bottom layers and cleaning the working chamber interior.

It is necessary to regulate air pressure in the working chamber by discharging air little by little while filling it with concrete, since the air pressure is increased gradually as concrete is placed. Excessive increases in the air pressure will cause a caisson to float upward while excessive decreases will develop water channels in the concrete.

The slump value of concrete must be determined with consideration given to the slump loss due to the increase of air pressure in the working chamber. As a guide, it is determined to be approximately 180 mm in general cases where the air pressure in the working chamber is lower than 0.29 MPa, and approximately 210 mm when the air pressure is 0.29 MPa or higher. In parallel, the mix proportion must be determined so as to prevent concrete material separation.

To prevent pipes from clogging during the placement of fill concrete, it is necessary to consider the pipe structures, such as use of pipes with fewer bended portions and use of equipment with less pumping resistance. It is also necessary to monitor the condition of the concrete in the chamber at all times and pay attention to conditions such as pumping pressure in pipes during concrete placement.

After completion of fill concrete placing, it is good to give the concrete pneumatic curing for 24 hours and shut off air after ascertaining the strength.

17.12 Bottom Slab of an Open Caisson

(1) Underwater concrete should be placed, in principle, by using a tremie or a concrete pump after confirming that there are no changes in the water level in a caisson.

(2) If the fill water in the caisson is to be removed after placing the bottom slab of concrete, the influence of uplift by groundwater shall be carefully examined.

Since reaction of bottom layers has to be transferred via a bottom slab to aside wall and a partition of a caisson main body in the open caisson method, earth and sand adhering to cutting edge interior and partition lower ends should be carefully removed by means of a rammer or a jetted water after completing the caisson-sinking process, so that no impurities may be left between placed bottom slab concrete and cutting edge or partition.

(1) It is desirable to have at hand tremies whose number is same as that equal to one tremie per at least 40 to 50 m^2 for a caisson without a partition or that equal to the number of partitioned containers for a caisson with partitions to place underwater concrete.

Underwater concrete should always be placed continuously, and a pipe top .end should always be penetrated in concrete to prevent cement paste from flowing out. And also during the placement of concrete, concrete volume and placement height of concrete should always be measured.

(2) After placing bottom slab concrete, when the ponding water is eliminated to build reinforced concrete slab above the bottom concrete, uplift by ground water may break the bottom slab and crack the caisson main body in an unfavorable case. Hence, elimination of the ponding water should be carefully reviewed to avoid such a negative impact.

[Source: Specification for Highway Bridges of Japan (Part IV Substructures)]

(5) Open Caisson Foundation

1) Construction Flow

i) Figure below show the construction flow of open caisson foundation. Checking of the bearing capacity of the bearing layer shall be executed directly after the excavation. Excavation shall be continued until the entire bottom surface penetrates sufficiently, as indicated in design documents. The depth of excavation shall be increased if there is not enough bearing capacity or the bearing layer does not appear at the design depth.

ii) Preparation shall be done at the site to clear the location of the caisson. If the surface layer is soft and is not able to bear the equipment, it shall be replaced.

iii) Mud, slime shall be removed very carefully prior to footing concrete casting in order to secure the design bearing capacity of the caisson foundation.

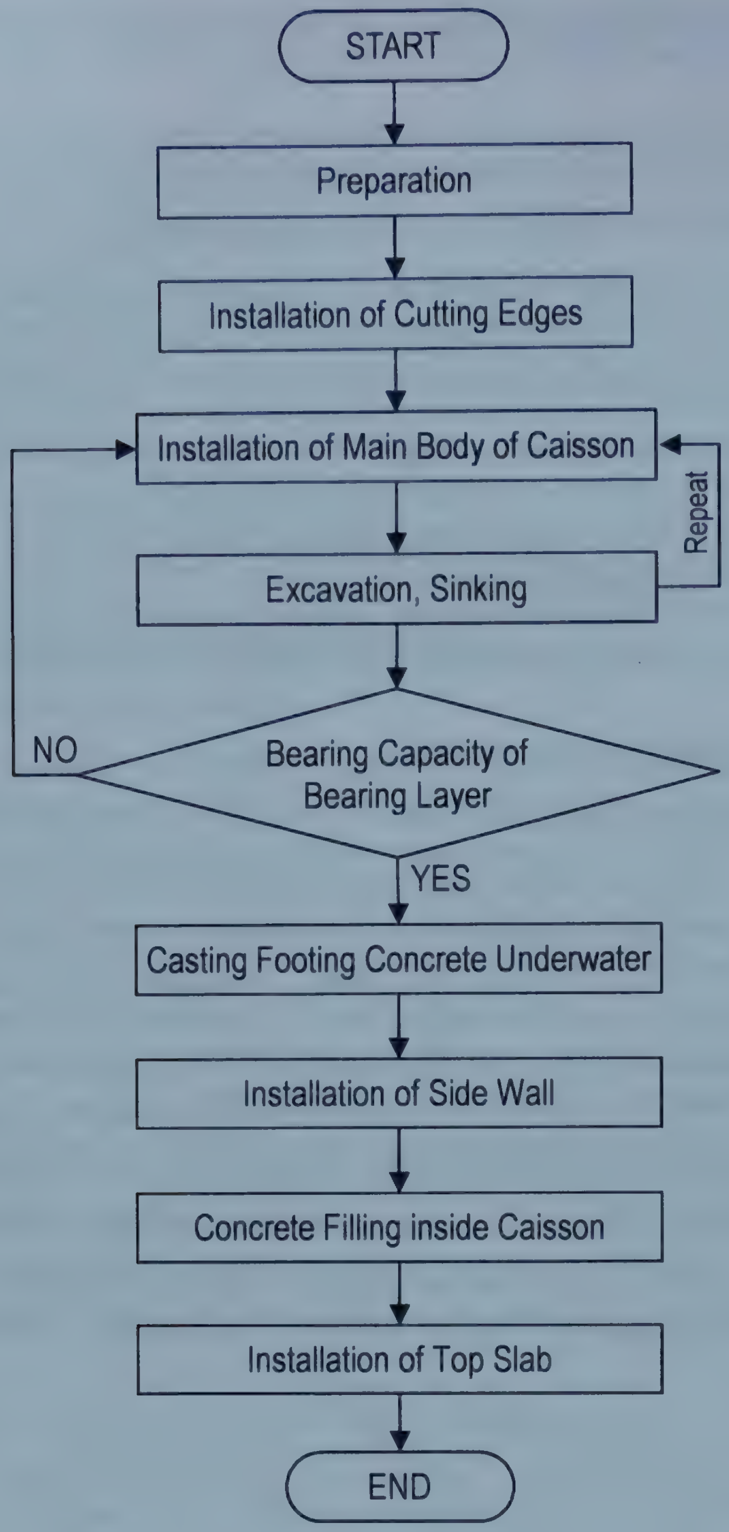

START
Preparation
Installation of Cutting Edges
Installation of Main Body of Caisson
Repeat
Excavation, Sinking
NO
Bearing Capacity of Bearing Layer
YES
Casting Footing Concrete Underwater
Installation of Side Wall
Concrete Filling inside Caisson
Installation of Top Slab
END

Figure 4-3: Construction Flow of Open Caisson Foundation

(6) Pneumatic Caisson Foundation

1) Construction Flow

i) Figure below show the construction flow of open caisson foundation. Checking of the bearing capacity of the bearing layer shall be executed directly after the excavation. Excavation shall be continued until the entire bottom surface penetrates sufficiently, as indicated in design documents. The depth of excavation shall be increased if there is not enough bearing capacity or the bearing layer does not appear at the design depth.

ii) Preparation shall be done at the site to clear the location of the caisson. If the surface layer is soft and is not able to bear the centre, it shall be replaced.

iii) Air pressure of working chamber shall be controlled to prevent water leaking from surroundings, it shall not be less than the pore pressure of the ground. However, if the air pressure is too strong, air blow to surrounding subsoil and loosen the subsoil very near to the caisson surface. Therefore, the pressure shall be monitored and controlled during the construction.

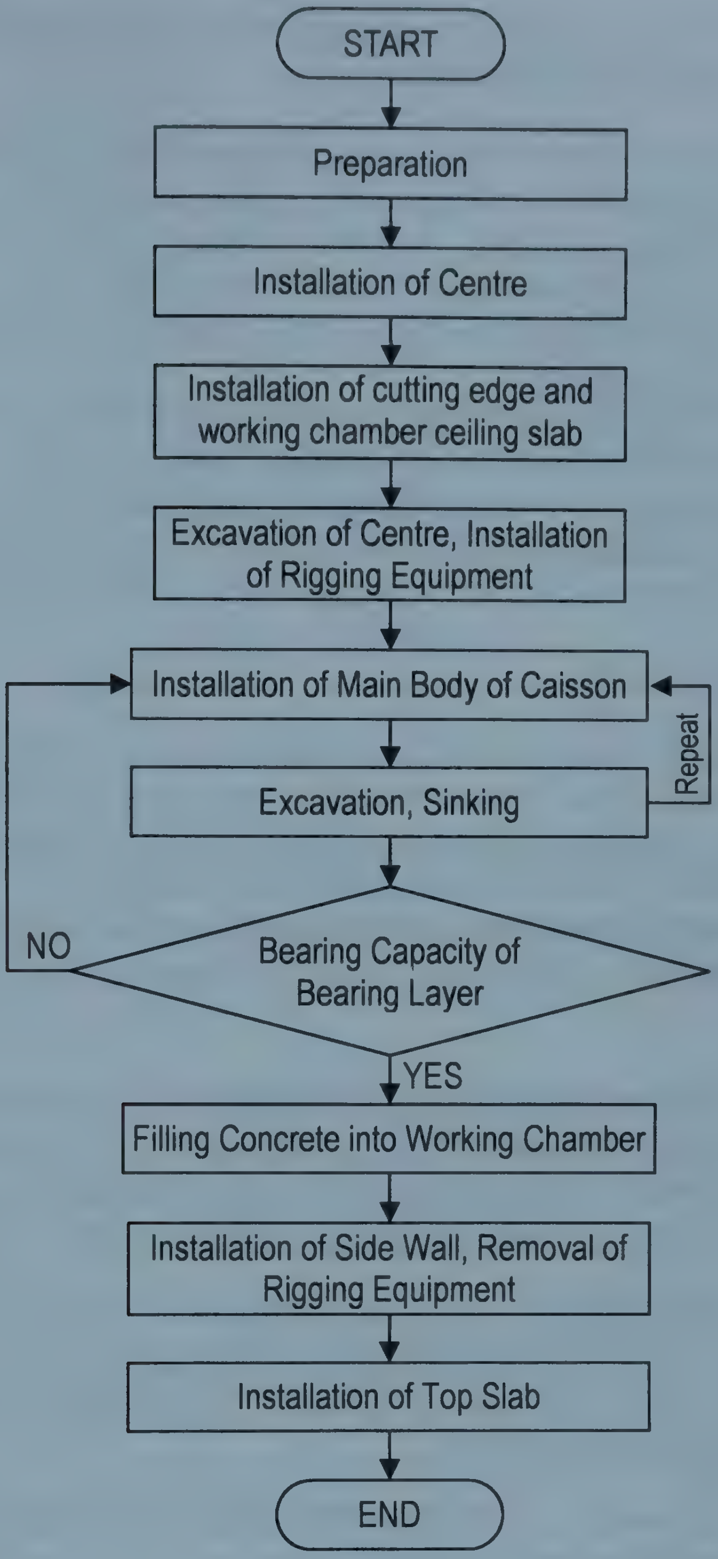

START
Preparation
Installation of Centre
Installation of cutting edge and working chamber ceiling slab
Excavation of Centre, Installation of Rigging Equipment
Installation of Main Body of Caisson
Excavation, Sinking
Repeat
NO
Bearing Capacity of Bearing Layer
YES
Filling Concrete into Working Chamber
Installation of Side Wall, Removal of Rigging Equipment
Installation of Top Slab
END

Figure 4-4: Construction Flow of Pneumatic Caisson Foundation

iv) High air pressure in working chamber does not allow labour to work in it for long period. It can be a cause of health problem for labour. Therefore, the working time of the labour in working chamber shall be controlled by individual both in accumulate and single time.

v) Mud, slime shall be removed very carefully prior to footing concrete casting in order to secure the design bearing capacity of the caisson foundation.

4.4 Substructure

4.4.1 General

4.4.1.1 Mountain bridges on mountainous and steep terrain usually have many constraints in accessing to the site unless the project is widening project of the existing route. It often have constraint to transport long or/and wide equipment or/and members even in the widening projects due to bad alignment and narrow width of the existing road. It mostly requires some amount of temporary road for its implementation.

Reducing the quantity of the temporary road contributes to reducing the cost and negative impact to the environment.

4.4.1.2 If the substructure construction site is located below the river level, there is limitation of season in constructing the substructure. The rainy season shall be avoided even it is urgent to implement the bridge. It is very difficult to secure the safety and also prevent the construction cost from increasing. Monitoring the precipitation and discharge shall be done anytime of the year to secure the safety.

4.4.1.3 Unlike the setting the design conditions, and design details, the most of the process in construction of substructure does not have many items which are specific to mountain bridges. However, there is high/very high RCC or hybrid piers which is mostly applied for mountain bridges and not much for other bridges. This subchapter treats the contents of construction of high/and very high piers mainly.

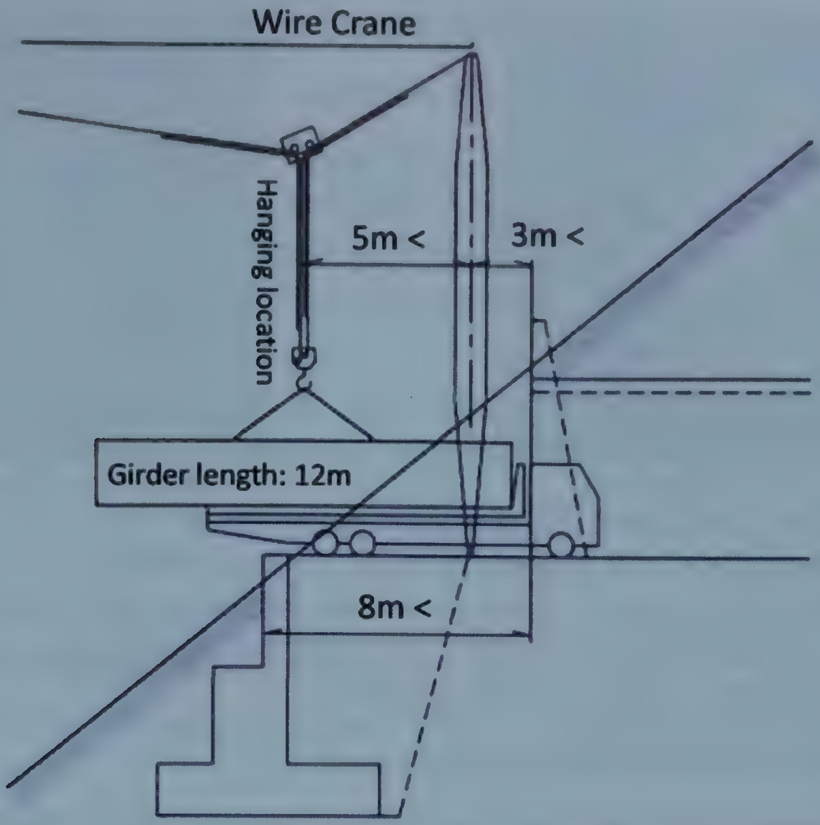
4.4.2 Abutment

4.4.2.1 If the project is implementing the bridge newly at the unexplored area, abutment could be constructed after the approach road construction has been commenced. Doing this prevents constructing temporary road to access the abutment sight. However, constructing long span bridges over deep valley or wide bridge, it is difficult to access the other abutment sight from different abutment sight following the bridge alignment. Temporary road or constructing road from both side of bridge is required.

4.4.2.2 Construction plan and schedule of abutments are closely related to the approach road construction as is mentioned in previous clause. Therefore, responsible bridge engineer shall keep close communication with road/highway engineer in considering the construction plan for bridge, specifically for abutments.

4.4.2.3 Abutments of river crossing bridge often face the risk of scour by flood flow. Specifically the rivers in India is influenced by monsoon and increase its discharge massively in short period. Therefore, the availability of inspection of scour after the opening shall be considered.

4.4.2.4 Abutment of mountain bridges could be placed at right beside the tunnel entrance. However, they shall not be stick together due to construction availability. Generally, it requires 8m of construction yard to construct the cast-in-situ pile foundation or spread foundation of the abutment. Applying stud/sloping abutment with deep foundation could be a solution to minimize this construction yard area and also reduces construction cost and negative impact to the natural environment.



(Source: Manual of Design and Construction of Road Bridges, Miyagi Prefecture)

Figure 4-5: Required Space for Erection of Girder in front of Tunnel Gate

4.4.3 Pier

4.4.3.1 Piers always requires temporary road to access the construction site unless there is a secondary road constructed parallel to the main bridge. However, most of mountain bridges have steep longitudinal gradient of the ground level along the alignment and it is often unrealistic to place the secondary road. If there are very steep slope along the alignment, the temporary road shall make long detour and requires massive amount of cut & fill to construction the temporary road and damages the nature. The temporary bridge or jetty can conserve nature and sometimes save the cost considering the cost of countermeasure of the natural environment. Therefore, planning the construction of pier including temporary road and structure shall be considered carefully before commencement of the construction.

4.4.3.2 Unlike the abutments, piers are often placed within the flood flow, and also river flow of low water level, no matter how the bridge engineer tries to avoid placing them. Severe condition of drainage of the river water from the site shall be set and executed to secure the safety of the site. However, rainy season shall be avoided if the substructure is located in flood flow area.

4.4.3.3 Pier within the flood flow, and also river flow of low water level, often face the risk of scour by flood flow. Specifically the rivers in India is influenced by monsoon and increase its discharge massively in short period. Therefore, the availability of inspection of scour after the opening shall be considered.

4.4.3.4 The typical structure type of pier is mainly three, such as wall type, column type, and rigid frame type. And the material is whether RCC or steel, but PSC is often applied when the beam of pier head is cantilevered with long span. However, steel and PSC is mostly applied for urban bridges, viaducts, and flyovers. And also rigid type are seldom applied either. Wall type and column type does not have any specific items for piers of mountain bridges unless it has height over 30m. Therefore, only high/very high piers with its height more than 30m is described in this guideline.

4.4.3.5 High/Very High RCC Piers

(1) Controlling the heat of hydration

1) Cross section of very high piers exceeds 100m^2 and the volume of the casting concrete would reach 500m^3 even the height of the casting form is about 3m high. The volume is very massive and produces remarkable heat of hydration which interfere the proper hardening process of cement concrete and will be the cause of thermal crack of the casted concrete.

2) By failing the control of heat of hydration, there will be considerable scale and number of thermal cracks which accelerate the deterioration of the concrete structure.

3) There are several countermeasures of preventing thermal crack, such as pre-cooling of concrete at batching plant and mixing vehicle, installing pipe cooling system in the cross section, casting in cool seasons, additional admixture to the ready mixed concrete, slowing down the temperature decrease by heat retaining, and/or installing crack induction joint at the surface.

(2) Inter-locking Rebar Arrangement

1) There is a unique method of rebar arrangement compared to conventional arrangement to reduce the procedure and speed up the construction. It is called "Inter-locking Rebar Arrangement Method" and is applied to high piers. This method was invented by NEXCO, Japan and applied to the piers in mountainous region for their bridge.

Extracted from Design Standard of NEXCO Japan, Part II, Bridge Construction, NEXCO

Chapter 5 Substructure

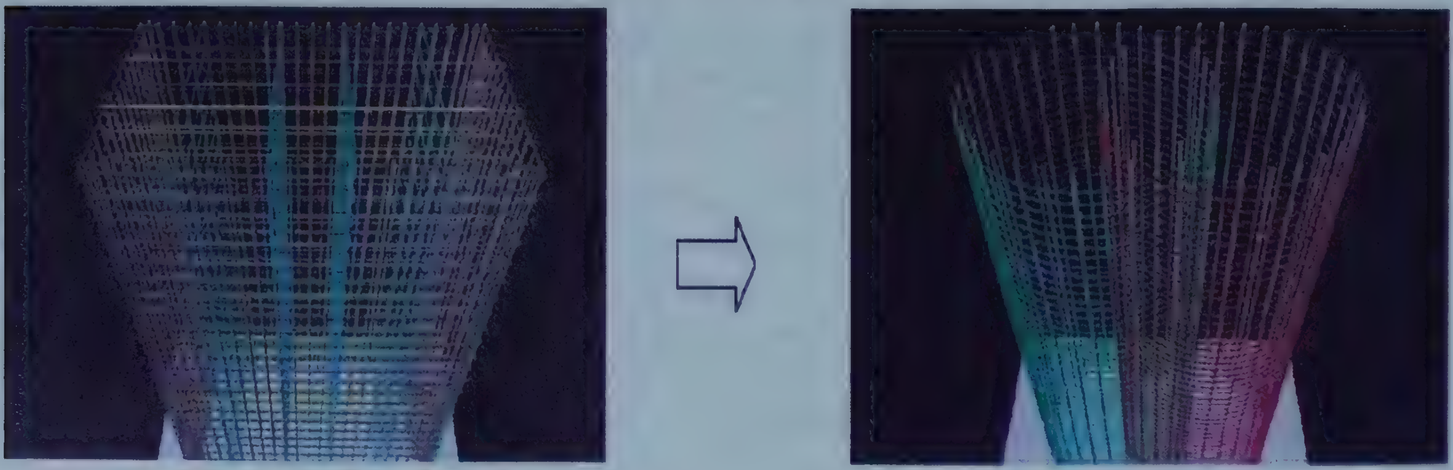
4. Pier

4-5 Pier with Inter-Locking Rebar Structure

4-5-1 Scope

This subchapter is applied to the RCC pier with inter-locking rebar arrangement structure.

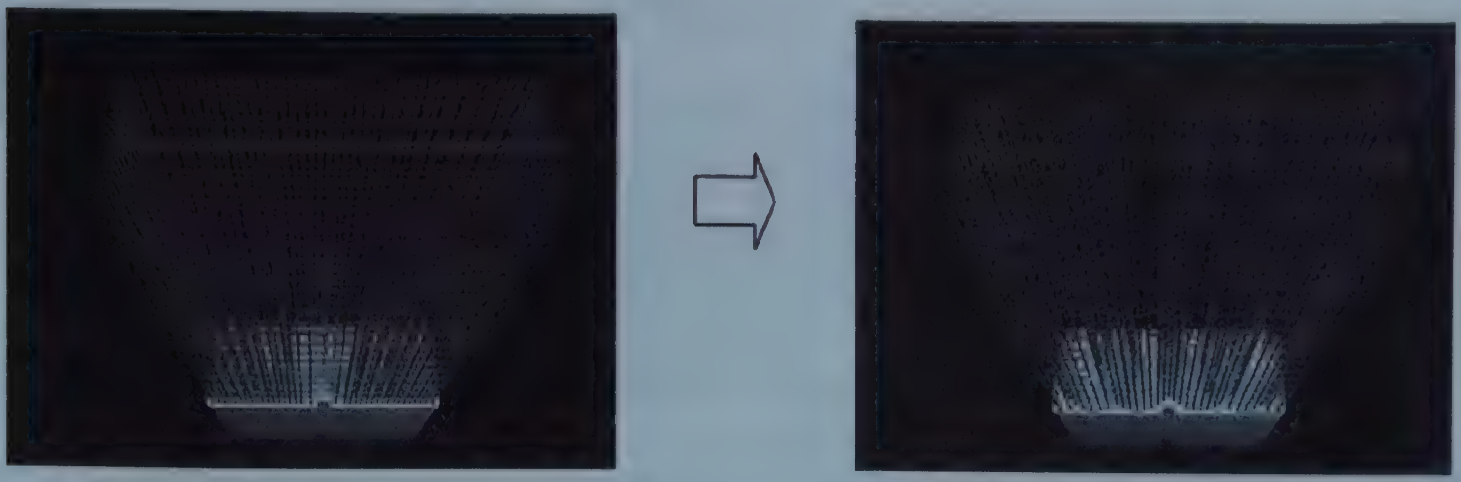
Pier with inter-locking rebar structure is invented in order to simplify the rebar arrangement in the typical cross section utilizing its superior aseismic capacity and efficient arrangement. Inter-locking rebar structure signifies the structure which has a rebar arrangement structure of several circular lateral ties (including spiral lateral ties) overlapping partially and react as horizontal restraint rebars in rectangular cross section without intermediate lateral ties. It secures high toughness by very high restraint effect of cement concrete by circular lateral ties even in a rectangular cross section (See Figure 5-3-40 and Figure 5-3-41). Spiral circular lateral ties can be applied to improve workability at site comparing the cost with conventional lateral ties.



Conventional Rectangular Rebar Arrangement

Double Inter-locking Rebar Arrangement

Figure 5-4-30 Rectangular Pier



Conventional Rebar Arrangement for Wall

Quadruple Inter-locking Rebar Arrangement

Figure 5-4-31 Wall Pier

4-5-2 Typical Cross section

(1) Interval of Centreline of Circular Lateral Ties

Interval of circular lateral ties next each other shall be set appropriately in order to transfer the shear in transversal direction.

(2) Arrangement of Axial Rebars

Axial rebars shall be arranged in order to transfer the shear born by circular ties considering the workability of assembling circular lateral ties on internal area of circular lateral ties connecting each other.

4-5-3 Design against Level 2 Earthquakes

(1) Horizontal Bearing Capacity and Displacement in Yield Stage and Horizontal Bearing Capacity in Ultimate Stage

Horizontal bearing capacity and displacement in yield stage and horizontal bearing capacity in ultimate stage shall be calculated according to the Specification for Highway Bridges V. Displacement in limit state of aseismic capacity 2 shall be set approximately considering the aseismic capacity of pier with inter-locking rebar arrangement.

(2) Shear Bering Capacity

Shear bearing capacity of inter-locking rebar arrangement structure shall be calculated according to the Specification for Highway Bridges V.

4-5-4 Review Method by Dynamic Analysis

(1) Basic Policy

Review method by dynamic analysis shall be referred to Chapter 3 of this standard and Specification for Highway Bridges V.

(2) Analysis Model

Skelton curve, reduction ratio of rigidity, and etc. of dynamic analysis shall be set appropriately according to the document in Chapter 3.7. “Modelling of Seismic Calculation” and considering the aseismic capacity of pier with inter-locking rebar arrangement

4-5-5 Review by Dynamic Analysis

(1) Circular Lateral Tie

Details of circular lateral ties, excluding spiral lateral ties, shall be referred to Specification for Highway Bridge V, Chapter 10.8.

(2) Spiral Lateral Tie

1) Vertical interval of spiral lateral tie shall be dense in plastic section of pier in resisting against earthquake. However, the interval shall be change gradually near the boundary of plastic section
2) Details of the connection and anchoring of spiral lateral tie shall be referred to the details of non-spiral circular lateral ties as mentioned in (1) above.

(3) Crack Prevention Rebar

Crack prevention rebars shall be arranged in order to prevent cover concrete cracking hazardously by drying shrinkage or temperature change at the section with large intervals of lateral ties.

---

(3) Composite Pier

1) There is also a method to construct high/very high pier using both RCC members and steel members as a composite structure. It is called "Composite Pier" and it has been improved defects and disadvantages of conventional RCC piers.

2) Conventional structure of high piers is RCC pier with massive amount of rebars with large diameter. It also has numerous numbers of ties (lateral tie and intermediate ties) to resist against shear. Therefore, rebar arrangement become very complicated and installing rebars before casting cement concrete had considerable numbers of procedures. It requires long period of construction and also becomes a reason of cost increase.

3) Composite pier utilizes larger steel members, such as steel pipes, H-beam, or I-beam, instead of rebars with large diameter as main member of the pier to resist against bending moment/tension force. I also utilizes spiral lateral ties to reduce numbers of rebars and simplifies the construction and also shorten the construction period.

4) Recently, precast forms are utilize to reduce even more pieces of members in construction of high/very high piers. Precast forms are fabricated with intermediate ties and buried to the main structure. It is sustained by larger steel members until casting cement concrete and it does not require other support structure for sustaining forms.

5) This clause introduces 3 method of composite pier with precast form which simplifies the construction procedure and also shorten the construction period. They are SPER method, REED method, and 3H method.

6) SPER Method

- **i)** SPER method is a rapid construction method of piers not only of high/very high piers but also for the average height piers. It utilizes precast panel with lateral and intermediate ties pre-installed and these panels forms the surface of the pier column. This precast forms with axial rebars and cement concrete casted inside forms composite structure.
- **ii)** Utilizing precast panels eliminates the use of support which was indispensable temporary structure in pier construction with conventional methods. It only requires light scale scaffolding and tower crane to set the precast panels. It downscales the facilities and equipment for construction.
- **iii)** This method was applied to the construction of Washimi Bridge in Gifu, Japan. One of the pier of Washimi Bridge is the highest pier among all the highway piers in Japan with height of 125m from the ground level.
- **iv)** SPER stands for "Sumitomo Mitsui's Precast Form for Earthquake Resistance and Rapid Construction".
- **v)** Figure below shows the precast panel of SPER method for rectangular column.

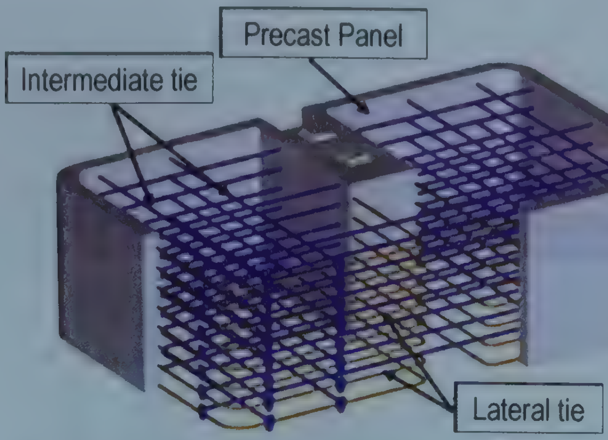


Figure 4-6: Precast Panel with Lateral and Intermediate Ties for SPER Method

vi) Figures below describe the construction procedure of pier by SPER method.

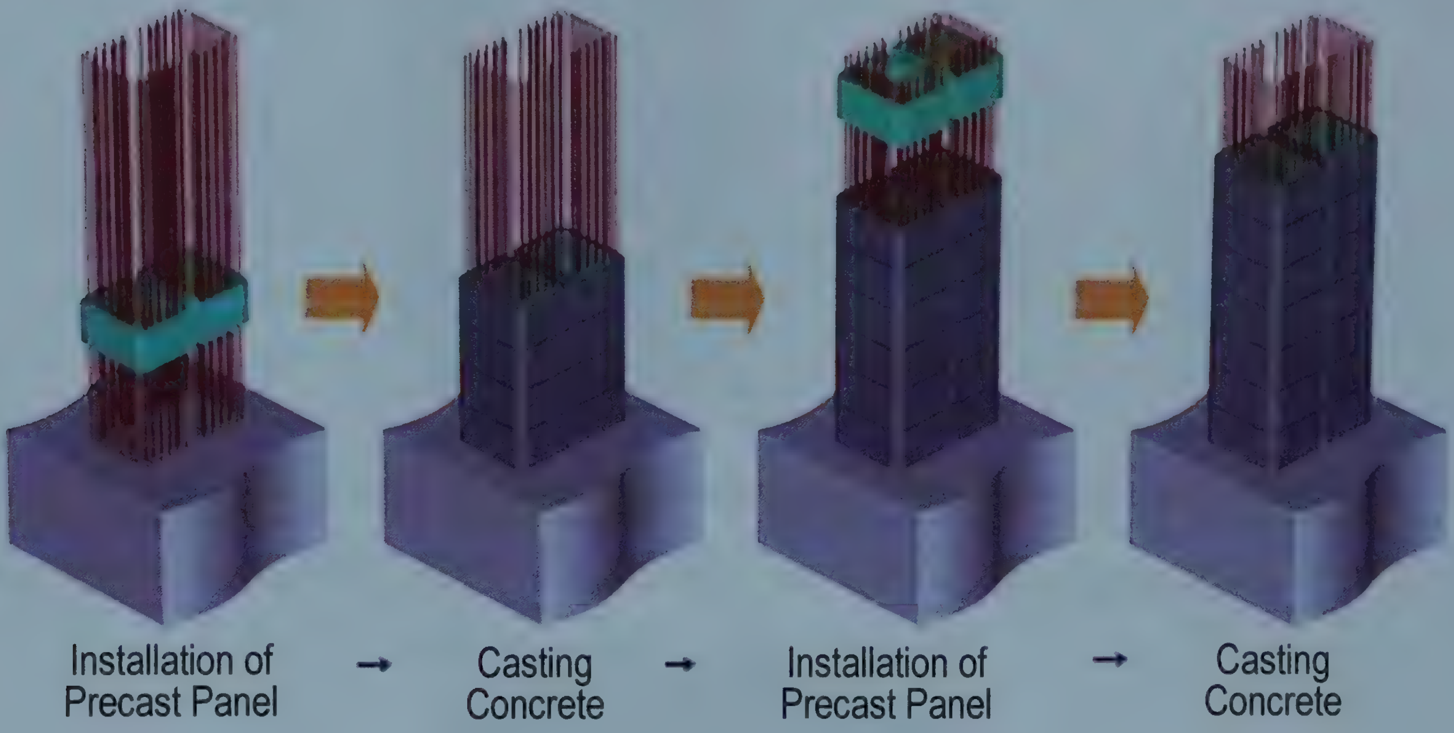


Figure 4-7: Construction Procedure of SPER Method

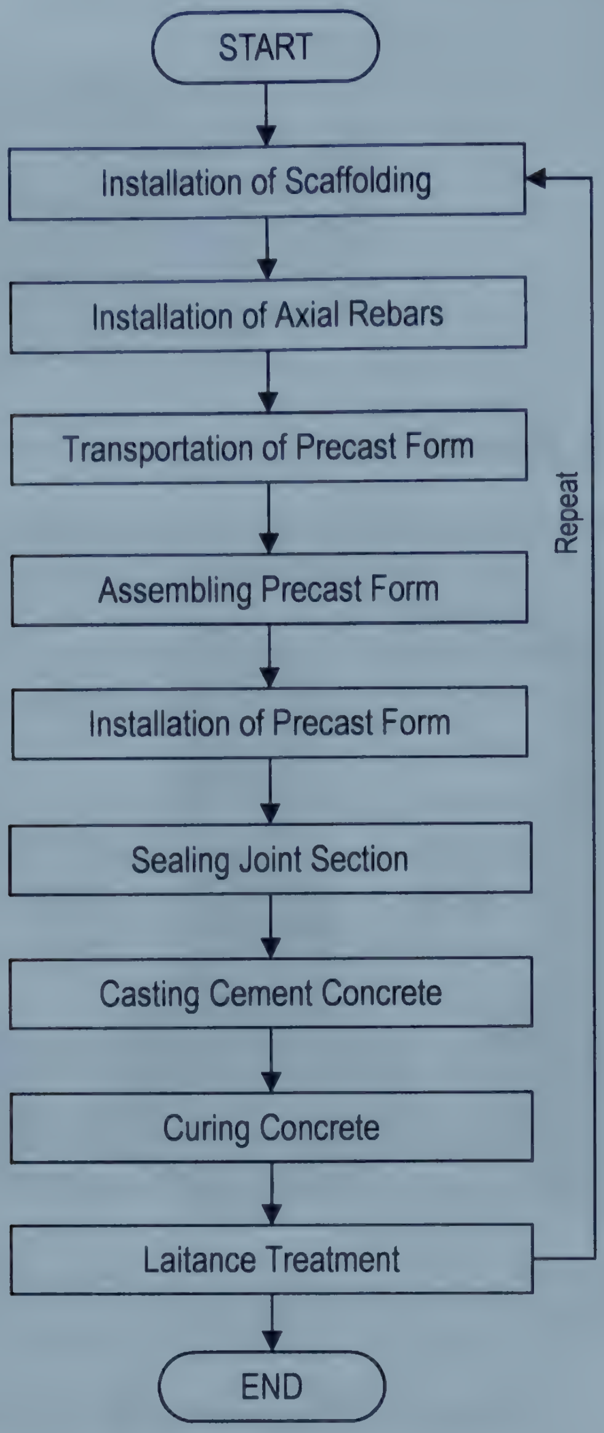


Figure 4-8: Construction Flowchart of SPER Method (1 lot, Height of 5m)

7) REED Method

i) REED method is a method of constructing composite pier of steel H-beam and cement concrete. It utilizes steel H-beam with projections in order to increase adhesive capacity with cement concrete. It also utilizes precast forms, with high durability called "SEED form", which form the surface of the column.

ii) SEED form has lateral and intermediate ties pre-installed and reduces the construction procedure at site considerably and shortens the construction period.

iii) H-beam with projections eliminate the use of axial rebars and also reduces the construction procedure at site considerably and shortens the construction period.

iv) Utilizing SEED form eliminates the use of support which was indispensable temporary structure in pier construction with conventional methods. It only requires light scale scaffolding and tower crane to set the precast panels. It downscales the facilities and equipment for construction.

v) Figure below shows the structure of REED method for rectangular column.

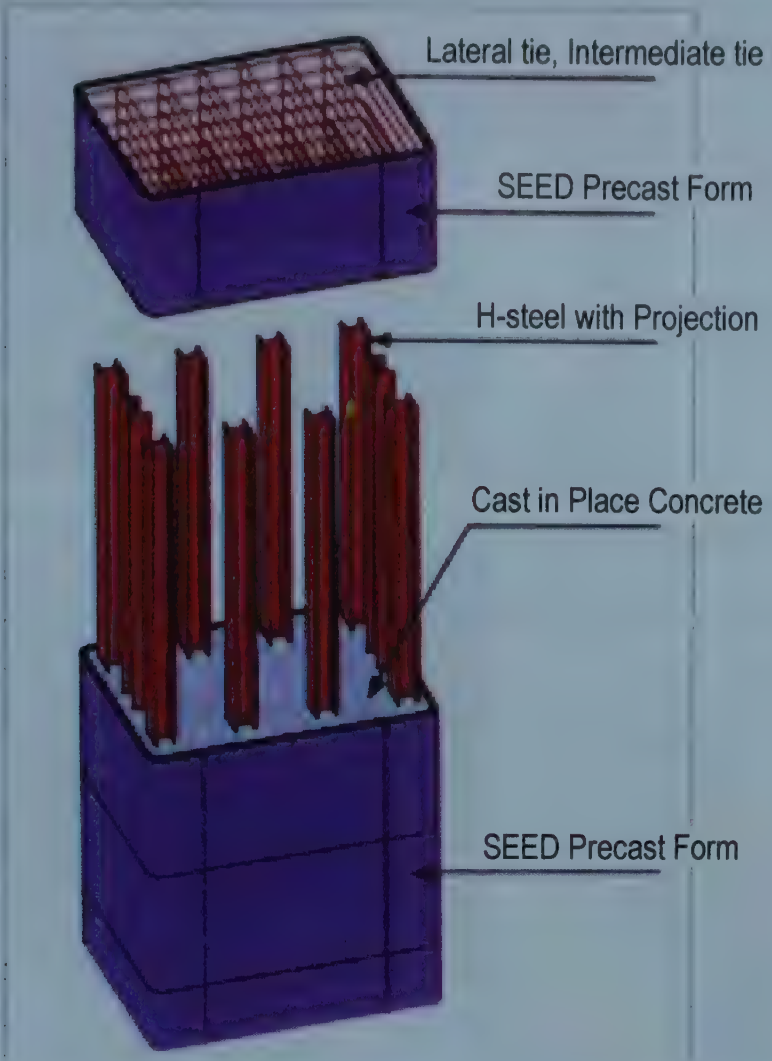


Figure 4-9: Structure of REED Method for Rectangular Column

vi) Figures below the precast panel of REED method for rectangular column.

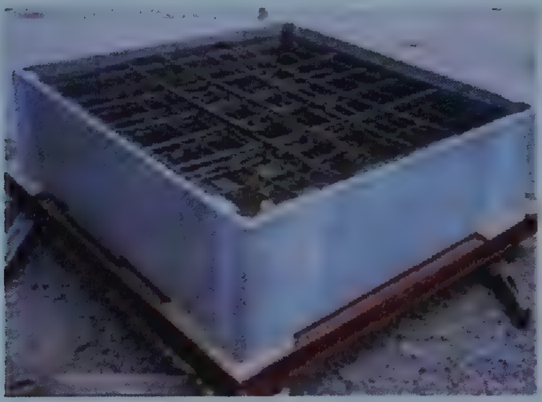
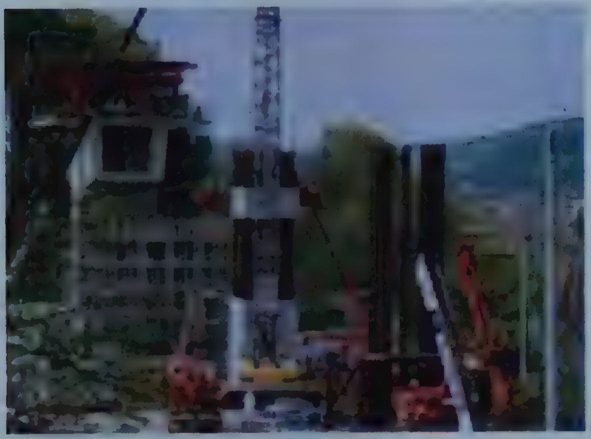


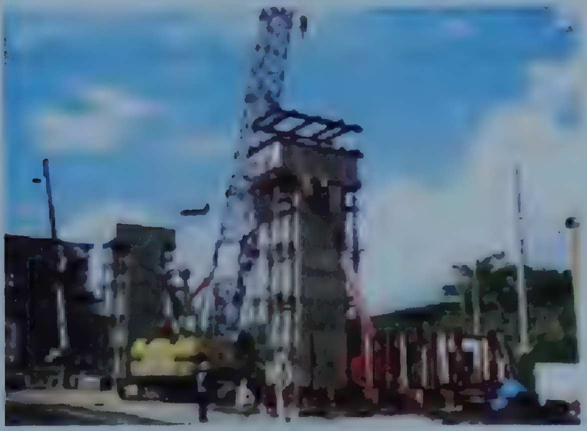
Figure 4-10: Precast Panel with Lateral and Intermediate Ties for REED Method

vii) Figures below describe the construction procedure of pier by REED method.

Installation of H-beam with projection



Installation of Pre-cast Panel (Column)



Installation of Pre-cast Panel (Beam)

Figure 4-11: Construction Procedure of REED Method

8) 3H Method

i) 3H method is a method of constructing high/very high pier applying prefabrication, hollow cross section, and hybrid structure in order to reduce the number and procedure of construction as well as secure high seismic capacity. It also shortens construction period and reduces construction cost. It can be applied to the pier height of 30 to 100m or more.

ii) Axial rebars of conventional pier is replaced by steel H-beam or I-beam wrapped around by spiral rebar. It is named "Spiral Column". It eliminates the needs of lateral and intermediate ties. It can reduce the construction procedure at site considerably and shortens the construction period.

iii) It also utilizes pre-cast panel called "3H panel" as form for casting concrete. It also reduces the construction procedure at site considerably and shortens the construction period. 3H panel forms the surface of the column.

iv) Utilizing 3H panel eliminates the use of support which was indispensable temporary structure in pier construction with conventional methods. It only requires light scale scaffolding and tower crane to set the precast panels. It downscales the facilities and equipment for construction.

v) Figure below shows the typical cross section of pier constructed by 3H method.

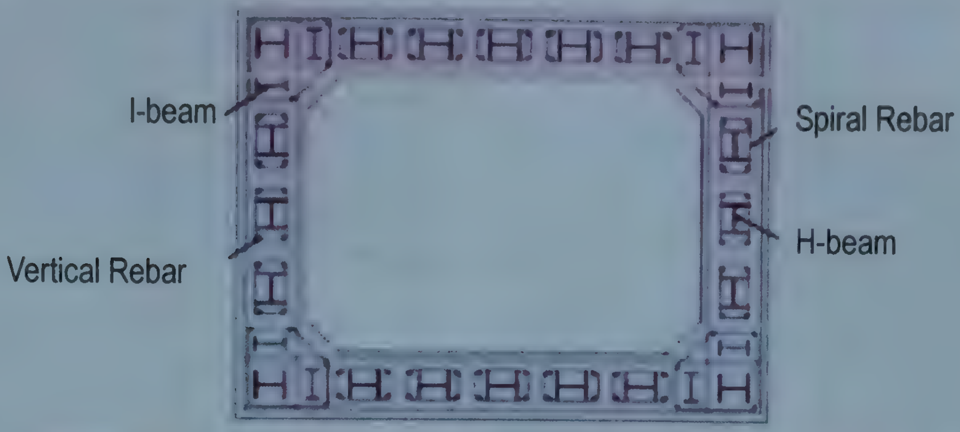
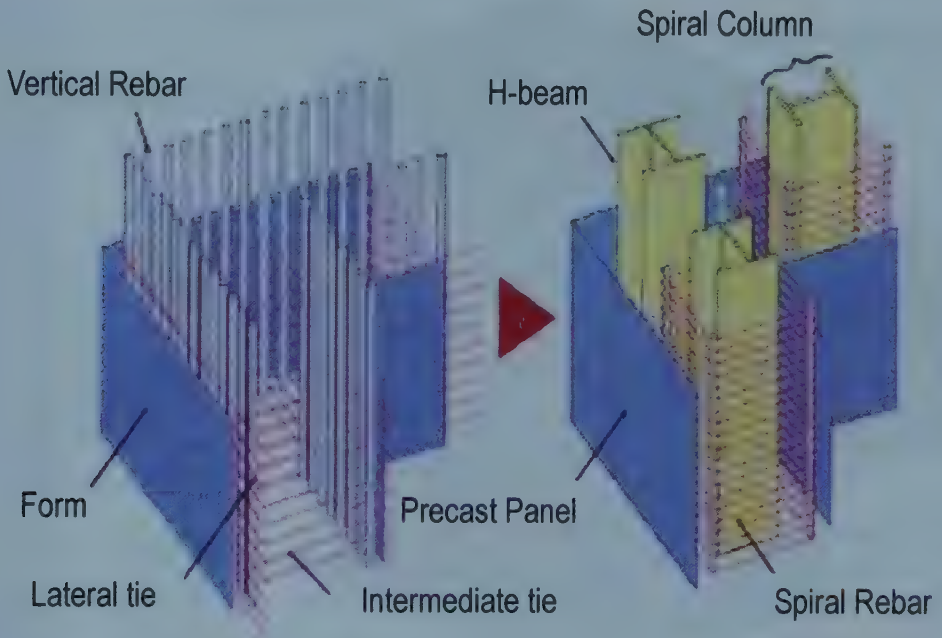


Figure 4-12: Typical Cross Section of Pier constructed by 3H Method

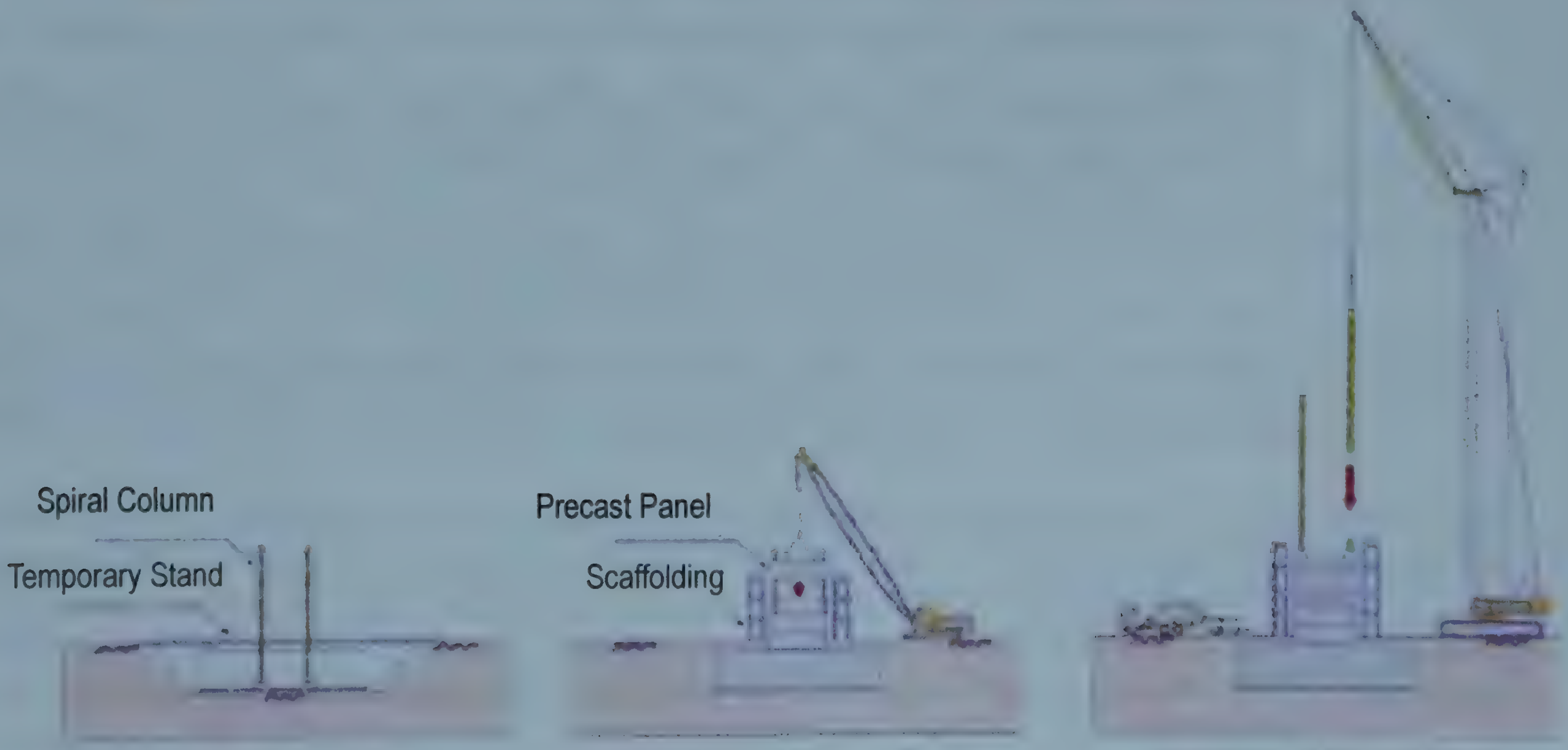
vi) Figure below shows the differences between conventional RCC pier and pier constructed by 3H method.



Conventional Pier Pier by 3H Method

Figure 4-13: Typical Cross Section of Pier constructed by 3H Method

vii) Figures below describe the construction procedure of pier by 3H method.



Installation of Spiral Column Installation of Precast Panel & Concrete Casting Installation of Precast Panel & Concrete Casting

Figure 4-14: Construction Procedure of 3H Method

(4) Self-Climbing Form

1) There require massive and very tall support to sustain form for casting concrete for high/very high piers in conventional construction method. Three construction method introduced in previous clause of composite pier utilizes precast panel/form to eliminate the use of conventional support.

2) There is another method to solve this problem. The solution is to apply self-climbing form sustained by the steel members of composite pier, such as steel pipe or steel H-beam.

3) Figure below shows the example of self-climbing form applied to 3H method. Self-climbing form is sustained by steel H-beam of the composite pier. The movement of the form is controlled by hydraulic jack connected and monitored by computing system. It improves safety, workability, and cost savings by shortening construction period.

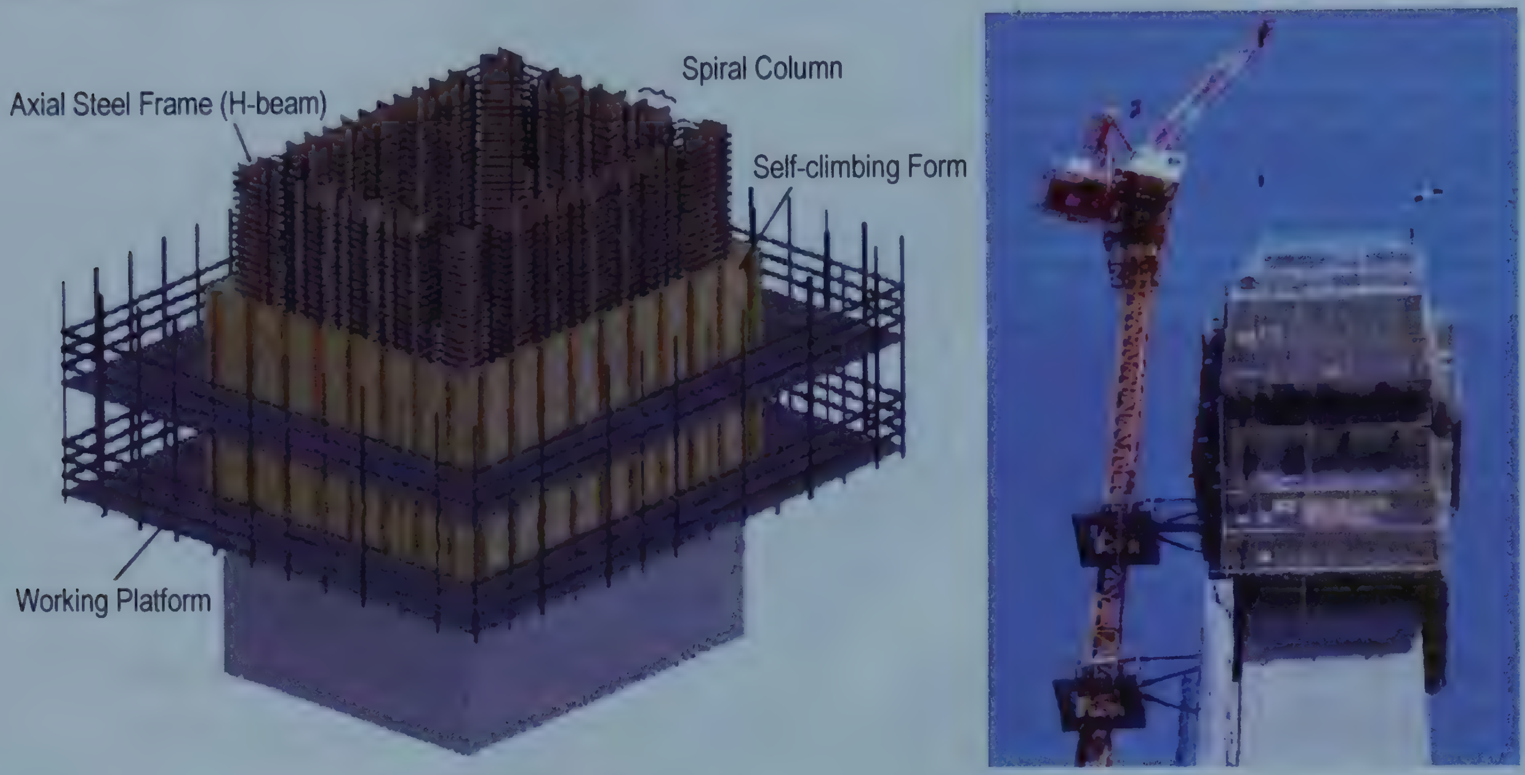


Figure 4-15: Self-Climbing Form applied to 3H Method

(5) Rigid Connecting with Superstructure

1) Span length of bridges in mountainous terrain tends to be longer than that of bridges in rolling/flat terrain due to its constraints to construct foundation/substructure. Therefore, many countermeasures to extend the span length is studied and applied in planning mountain bridges.

2) One of the countermeasures to extend the span length is connecting superstructure and substructure rigidly and form integral bridge. It can be applied to many types of superstructure, such as PSC box-girder bridge, composite superstructure (corrugated steel web bridge, composite truss bridge, etc.), and even with steel girder bridge. It can also be applied to extra-dosed bridge or other types of superstructure with outer cable prestressed structure.

3) Recently integral bridge with steel girder superstructure is developed and called composite integral (rigid framed) bridge. Its superstructure is continuous steel plate girder and substructure is RCC pier. Superstructure has cross beams and diaphragms at the pier cap section and cast-in-situ cement concrete connects both members rigidly. Volume of cement concrete casted at site becomes massive and require heat treatment (mass concrete treatment).

4) Figure below show the structure and example of composite integral (rigid framed) bridge.

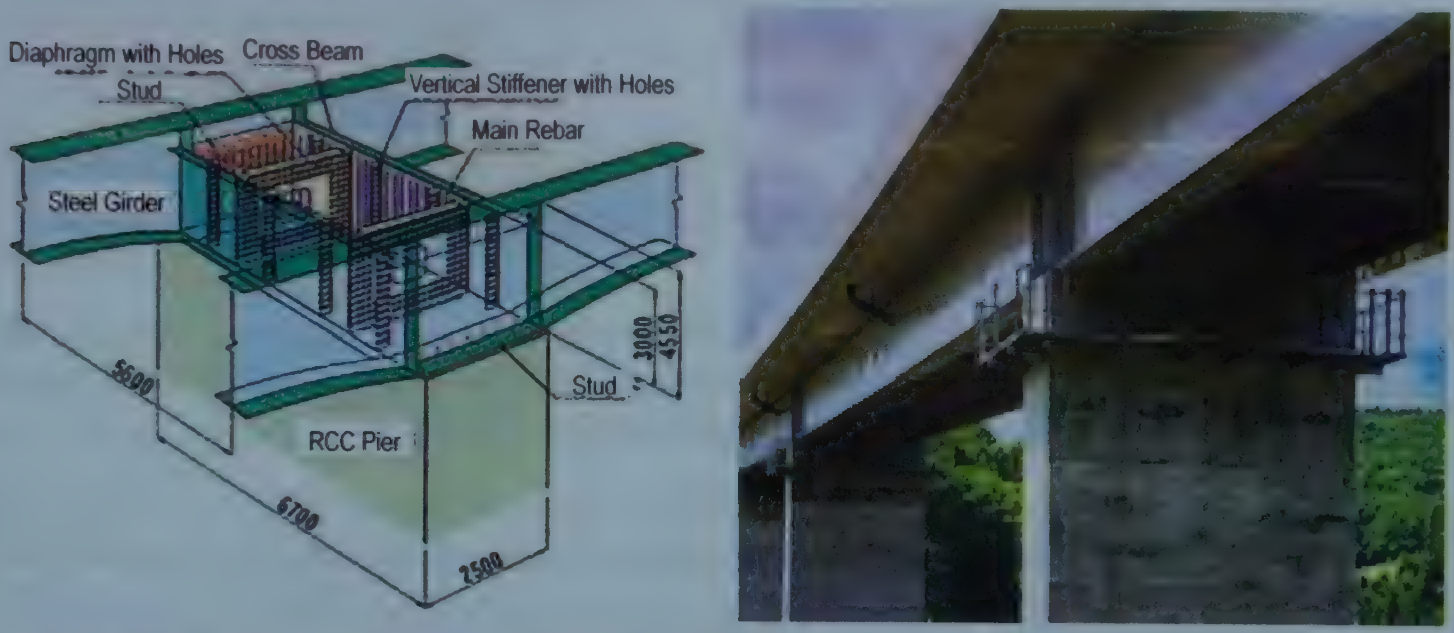


Figure 4-16: Structure and Example of Composite Integral (Rigid Framed) Bridge

4.5 Superstructure

4.5.1 General

4.5.1.1 Cheapest superstructure for mountainous bridges is also full-staging cast-in-situ concrete superstructure just as the bridges in other terrain. However, full-staging for casting concrete at site is usually not possible or costs very high for mountain bridges due to limited location of substructure construction and high space beneath the superstructure considering terrain condition. Therefore, other types of superstructures have advantages in mountainous and steep terrain.

4.5.1.2 Most of the major mountain bridges have longer span length than 50m due to the constraints of substructure constraint. The span length can reach over 200m in crossing deep valley. Therefore, types of superstructure which are able to apply are limited. PSC box section, PSC extra-dozed, steel box girder, Arch, steel orthotropic slab, cable stayed, and suspension bridge. Some of the steel-concrete composite superstructure is increasing in recent years, such as corrugated steel web, composite truss, and mixed steel/PSC superstructure. Balanced cantilever erection method or erection by wire crane is most for these types of superstructure construction.

4.5.1.3 In case of minor mountain bridges, there is not much difference in erecting superstructure with bridges in other terrain. However, procurement of materials, equipment and facilities are very different in terms of cost, transportation, and storage. This aspect is not specific for superstructure, therefore construction of superstructure of minor mountain bridges are extracted from this guideline.

4.5.1.4 Applying weathering/weatherproof steel for mountainous bridge, special care shall be taken care to form stabilized rust on the surface of member from fabrication, transportation, storage, and erection. Details is described in clause 4.5.5 below.

4.5.1.5 IRC codes for construction of superstructure, there are two codes to be referred to. There are IRC SP 065:2005 'Guidelines for Design and Construction of Segmental Bridges' and IRC SP 071:2006 'Guidelines for Design and Construction of Pretensioned Girder of Bridges'. However, both superstructure types are not applied often for mountain bridges. It is applied to long length viaducts in flat terrain more commonly.

4.5.2 PSC Superstructure

(1) PSC box girder

1) Construction of PSC box girder of mountainous bridges does not have very different features compared to the bridge in other terrain. However, span length shall be longer than bridges in other terrains and girders tends to be continuous and rigidly connected with substructure.

2) Most likely the erection method is balanced cantilever method. Superstructure shall have inner/outer PSC cables installed. Details of erection shall be referred to IRC 112 2011 Code of Practice for Concrete Road Bridges.

(2) PSC extra-dosed

1) PSC extra-dosed superstructure is another solution to extend span length.

2) Figure below show the construction procedure of extra-dosed bridge.

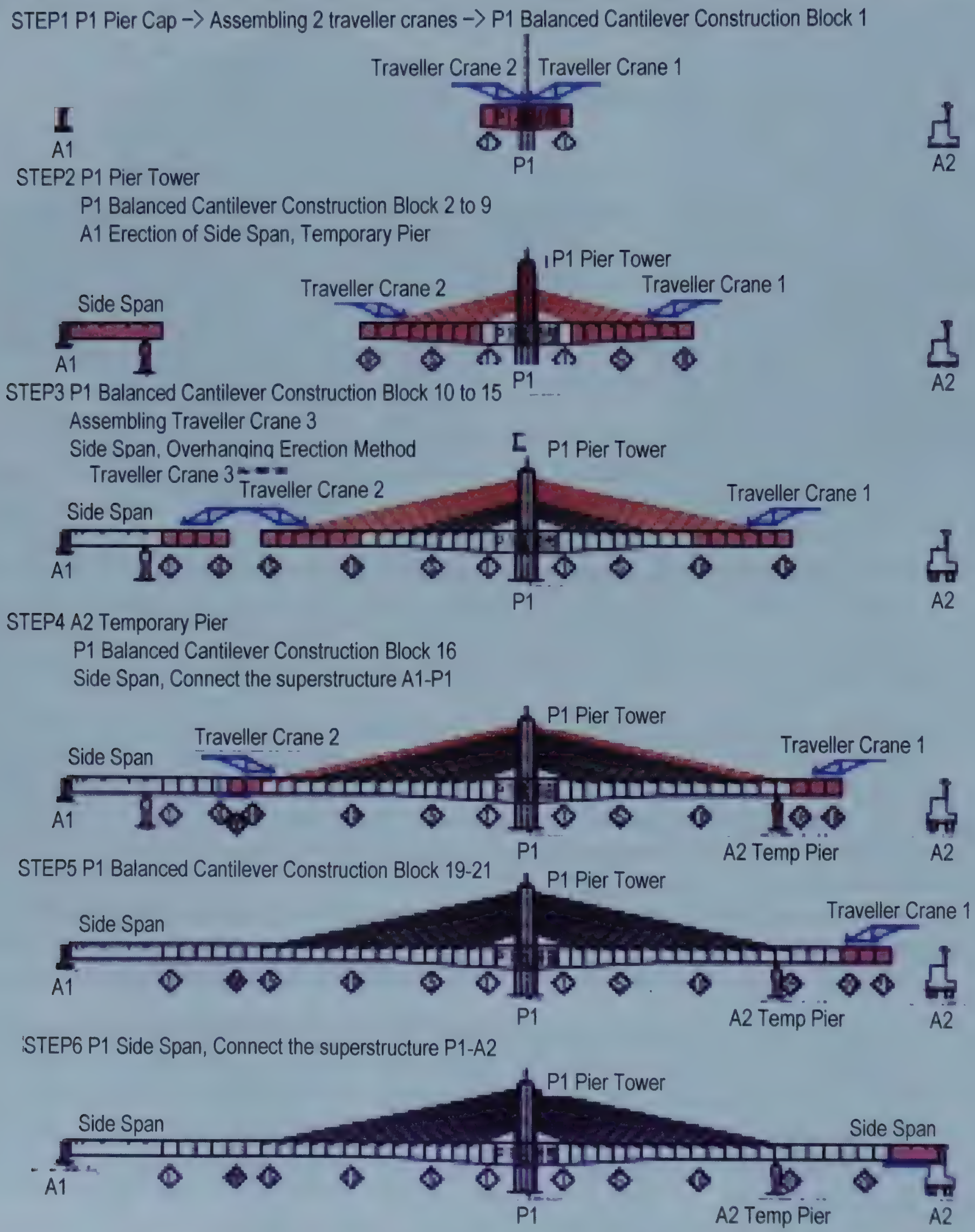

STEP1 P1 Pier Cap –> Assembling 2 traveller cranes –> P1 Balanced Cantilever Construction Block 1
Traveller Crane 2
Traveller Crane 1
A1
P1
A2
STEP2 P1 Pier Tower
P1 Balanced Cantilever Construction Block 2 to 9
A1 Erection of Side Span, Temporary Pier
P1 Pier Tower
Side Span
Traveller Crane 2
Traveller Crane 1
A1
P1
A2
STEP3 P1 Balanced Cantilever Construction Block 10 to 15
Assembling Traveller Crane 3
Side Span, Overhanging Erection Method
P1 Pier Tower
Traveller Crane 3
Traveller Crane 2
Traveller Crane 1
Side Span
A1
P1
A2
STEP4 A2 Temporary Pier
P1 Balanced Cantilever Construction Block 16
Side Span, Connect the superstructure A1-P1
P1 Pier Tower
Traveller Crane 2
Side Span
Traveller Crane 1
A1
P1
A2 Temp Pier
A2
STEP5 P1 Balanced Cantilever Construction Block 19-21
P1 Pier Tower
Side Span
Traveller Crane 1
A1
P1
A2 Temp Pier
A2
STEP6 P1 Side Span, Connect the superstructure P1-A2
P1 Pier Tower
Side Span
Side Span
A1
P1
A2 Temp Pier
A2

Figure 4-17: Construction Procedure of Extra-dosed Bridge

4.5.3 Steel Superstructure

(1) Steel box girder

1) Details of erection shall be referred to IRC 024 2010 Standard Specifications and Code of Practice for Road Bridges, Section V Steel Road Bridges (Limit State Method).

(2) Arch Bridge

1) Details of erection shall be referred to IRC 024 2010 Standard Specifications and Code of Practice for Road Bridges, Section V Steel Road Bridges (Limit State Method).

(3) Steel orthotropic deck

1) Unlike the RCC decks of which cement concrete is casted at site, steel orthotropic deck is prefabricated and assembling is the main works at site. Therefore, quality control of fabrication process is very important in construction of steel orthotropic deck. Welding shall be taken serious care since there are serious damages found on the existing steel orthotropic decks in Japan.

2) Consideration and cautions which shall be taken in executing works at site is explained below. The contents are quoted from "Handbook on construction of Steel Road Bridges, Japan Road Association".

Quoted from Handbook on construction of Steel Road Bridges, Japan Road Association

Section III Construction Works at Site

Chapter 3 Erection

3.3.8 Considerations on Erection by Superstructure Type

(10) Steel Orthotropic Deck

Followings shall be considered and treated specially.

1) Hanging

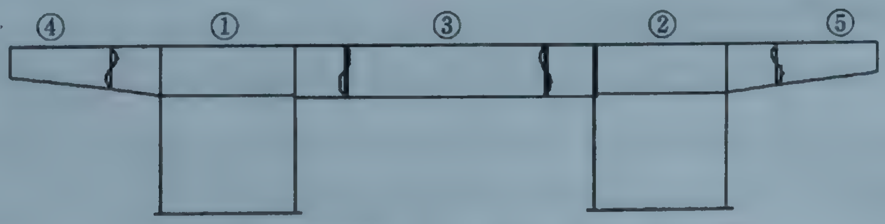
i) Deformation of each member caused by hanging shall be minimized by executing countermeasures, such as using balancing devices, since the deformation of steel orthotropic deck is much larger than other types of members.

ii) Hanging hook shall be installed in fabrication to prevent damage by sling wire.

2) Jointing

i) Final bolting of high tensile bolts shall be executed in hours with small effect by temperature difference within member due to sunshine to prevent sledge and torsion.

ii) Location, height, and gap of/with main girder shall be checked very carefully when jointing centre deck and deck slab. Specifically in the case when the main girder was erected without support of temporary bents. Deflection of main girder could differ considerably from design value caused by the effect of sunshine, deflection of main girder and erection equipment.



Procedure: ①→②→③→④→⑤

Figure III.3.3.15 Erection Procedure of Steel Orthotropic Deck

iii) If there are welding works at site, there shall be margin to handle the shrinkage caused by welding in fabrication. Shrinkage amount differs by support condition of main girder, welding method/equipment, welding procedure. When there is welding line in longitudinal direction, assembling and erection shall be done considering the shrinkage at this longitudinal welding line to secure the width of carriage way, for instance.

iv) Displacement of opening of welding line due to welding shrinkage changes by welding procedure when executing the entire cross section welding. Therefore, the displacement of the checking points between the end of the opening and the starting point of welding shall be design stays within the allowable range. Welding procedure, supporting device and locations shall be closely considered. Subchapter 3.5.7 Welding Works at Site shall be referred to for the details.

v) Final bolting shall be executed after welding works is completed when web plate/bottom flange is connected to member which are to be welded. Bolts shall be bolted temporarily while welding.

[Source: Handbook on construction of Steel Road Bridges, Japan Road Association]

3) Simplified Steel Orthotropic Deck

i) Figure below show the average effect of applying simplified steel orthotropic deck compared to steel orthotropic deck with conventional structure. There is a report which mentions that numbers of member (large/small), total welding length, total area of painting is reduced more than 40%.

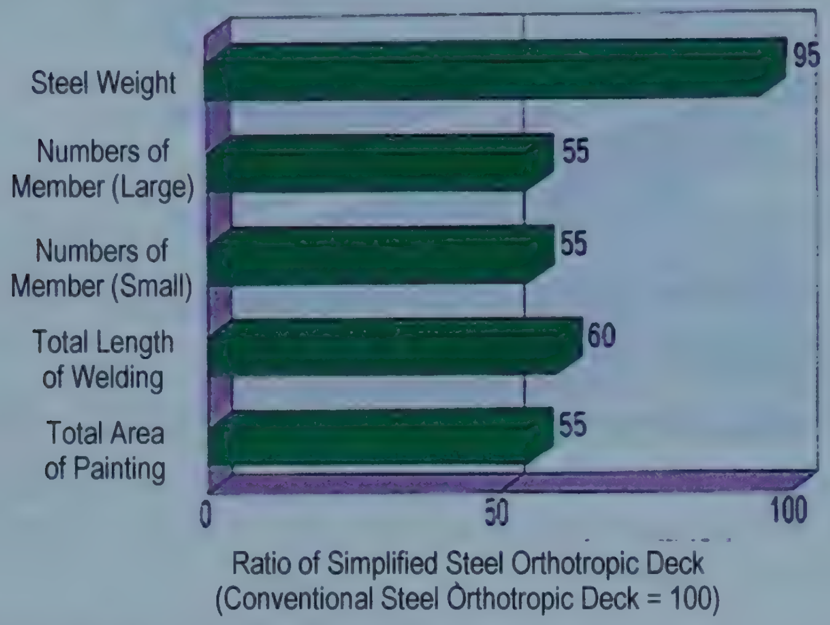


Figure 4-18: Effect of Simplification of Steel Orthotropic Deck

ii) Reduction of total are of painting contribute considerably to decreasing of Life Cycle Cost (LCC) as well. Simplification of structure contributes to improving the easiness of inspection. It significates that the damage can be found in earlier stage which is much easier to apply preventive repair methods. Applying preventive repair method also contributes to reducing of LCC.

4.5.4 Composite Superstructure

(1) Corrugated steel web

1) Corrugated steel web superstructure is another solution to extend span length.

2) Figure below show the construction procedure of corrugated steel web bridge.

i) Step 1: Construction of Pier Cap Section and Start Balanced Cantilever Blocks

Superstructure shall be connected rigidly with pier at pier cap section. There are cross beam, diaphragms, and also anchorages of outer cable and it will be very complicated. However, volume of casting concrete is massive and heat treatment shall be taken. Filling of concrete shall be carefully controlled despite complicated structure.

ii) Step 2: Construction of Balanced Cantilever Blocks

Construction procedure of single cantilever blocs is as follows,

I) Installation of scaffolding and form for bottom slab
II) Set corrugated steel web, arrange rebar and PSC cable for bottom slab
III) Cast cement concrete for bottom slab
IV) Installation of form for top slab
V) Arrange rebar and PSC cable for top slab
VI) Cast cement concrete for top slab
VII) Provide tension outer-cable
VIII) Remove scaffolding and form and shift to next block

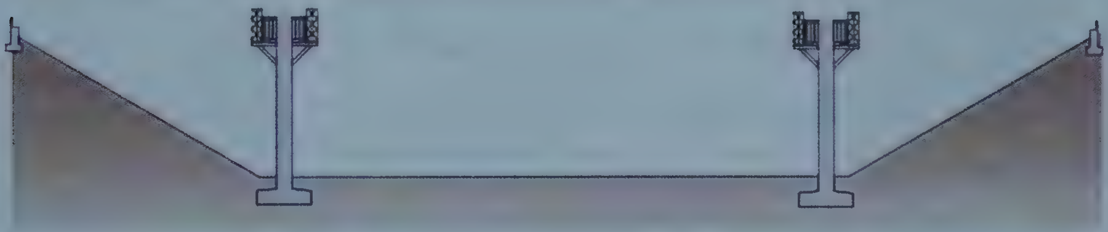
If PSC slab or composite slab is applied for top/bottom slab, it can reduce the construction period even more.

iii) Step 3: Connecting Centre of Span Rigidly and Finish Side Spans

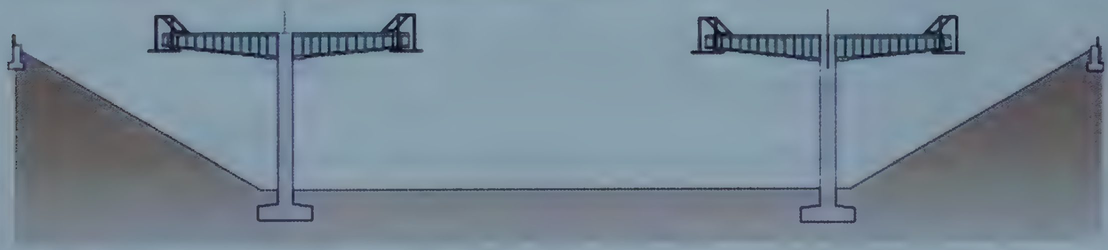
Centre of span shall be connected rigidly and provide tension by PSC cable.

End of span shall be placed on the bearings.

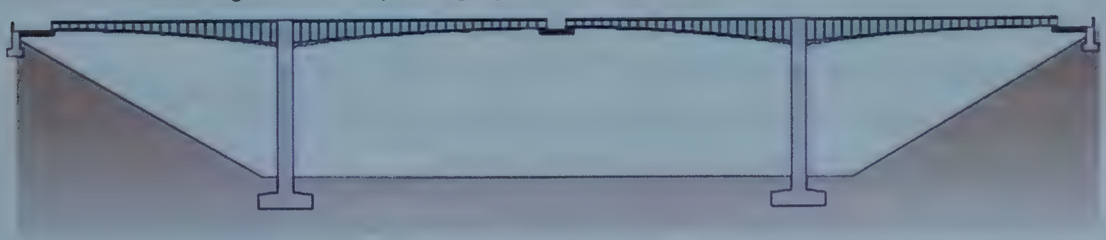
STEP 1 Construction of Pier Cap Section and Start Balanced Cantilever Blocks



STEP 2 Construction of Balanced Cantilever Blocks



STEP 3 Connecting Centre of Span Rigidly and Finish Side Spans



STEP 4 Surface Works

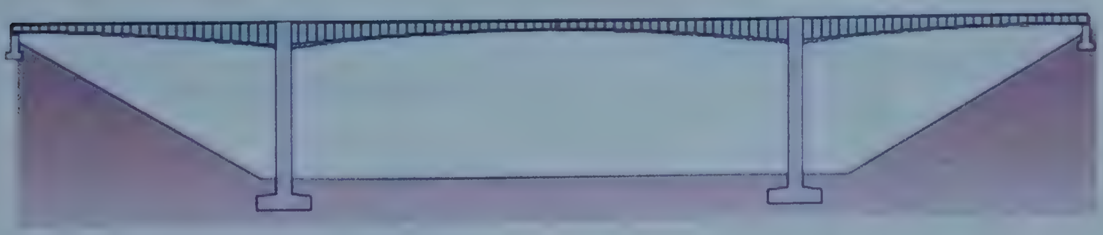


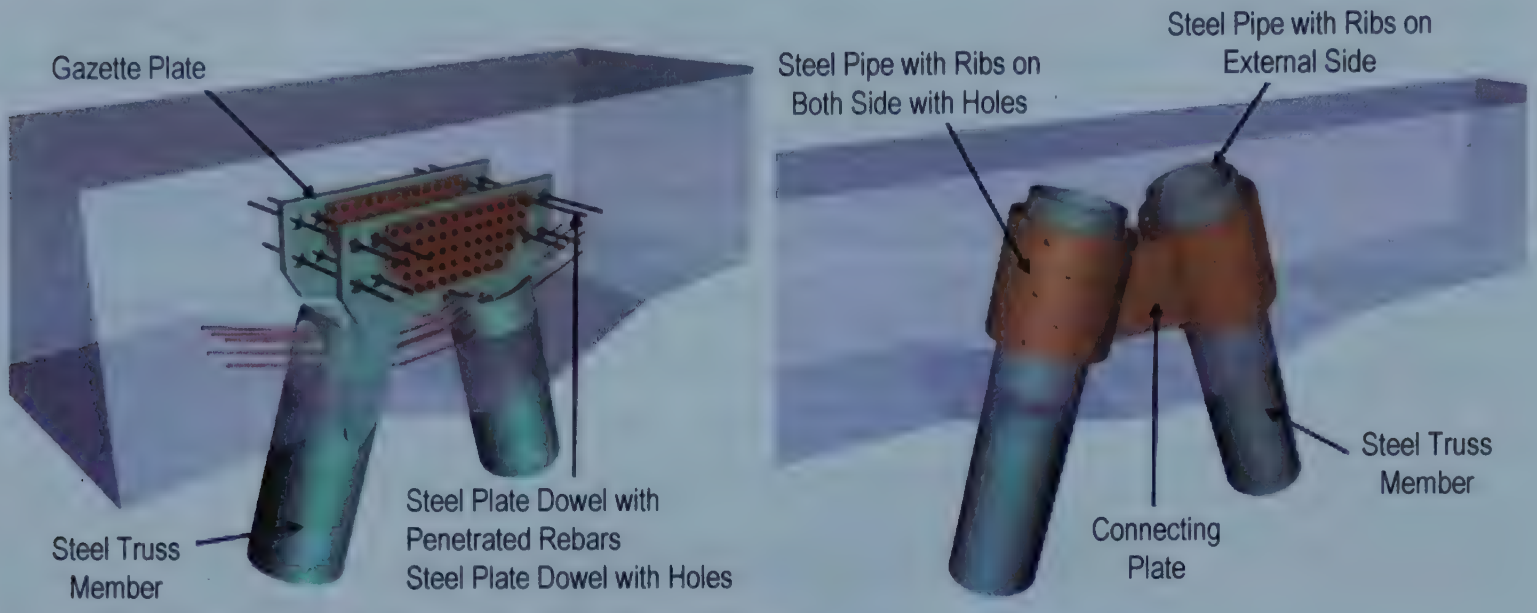
Figure 4-19: Construction Procedure of Corrugated Steel Web Bridge

(2) Composite truss

1) Composite truss superstructure is another solution to extend span length.

2) Construction procedure of Composite truss bridge is very similar to that of corrugated steel web bridge. The steel web is replaced to steel truss members in composite truss bridge.

3) Unlike the corrugated steel web, connectivity between steel truss members and concrete slabs shall be confirmed by detailed analysis in design stage. Figure below shows two examples of typical structure of connecting section with top slab. Both method has several application facts.


Gazette Plate
Steel Truss Member
Steel Plate Dowel with Penetrated Rebars
Steel Plate Dowel with Holes
Steel Pipe with Ribs on Both Side with Holes
Steel Pipe with Ribs on External Side
Steel Truss Member
Connecting Plate

Double Gazette Type Double Pipe Type

Figure 4-20: Examples of Typical Structure of Connecting Section of Steel Truss Member and Top Slab

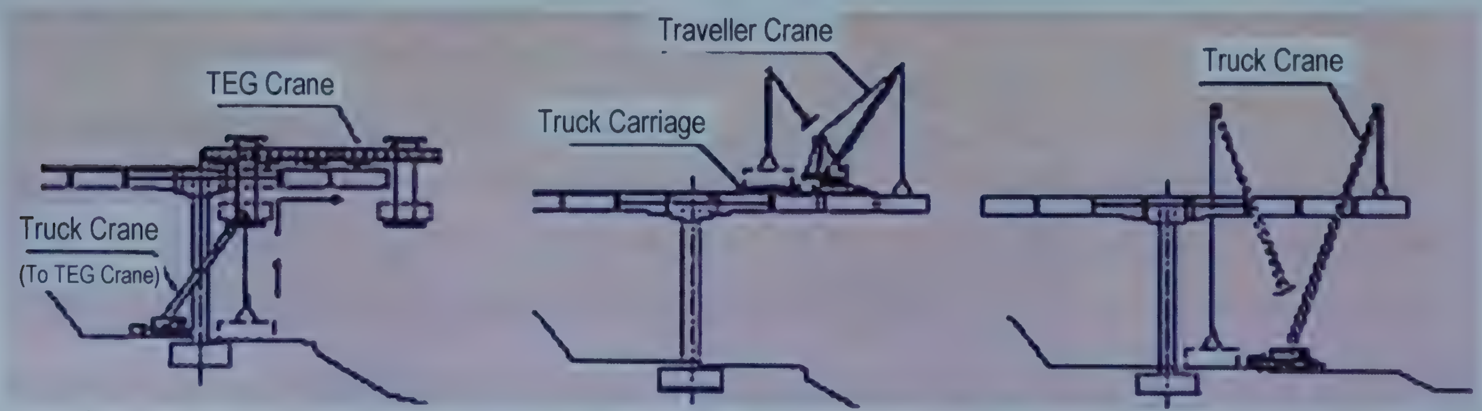
(3) Composite Integral (Rigid Framed) Bridge

1) Composite integral (rigid framed) bridge is another solution to extend span length.

2) Figure below shows the construction/erection methods which can be applied for composite integral bridge.

I) Overhanging Erection Method with TEG Crane
(TEG: Travelling Erection Gantry)
II) Overhanging Erection Method with Traveller Crane
III) Overhanging Erection Method with Truck Crane

III) is not easily applicable to the bridge on steep slope. I) and II) is most likely to be suitable erection method. TEG crane has advantage in running cost and traveller crane in initial cost. If the bridge length is long TEG crane has more advantage than traveller crane, but vice versa for short bridges.



Overhanging Erection Method with TEG Crane
Overhanging Erection Method with Traveller Crane
Overhanging Erection Method with Truck Crane

Figure 4-21: Erection Method of Composite Integral (Rigid Frame) Bridge

3) Figure below show the construction procedure of composite integral (rigid framed) bridge. It shall be taken good use of procedures of construction works at site and fabrication of steel member at factory can be executed in parallel.

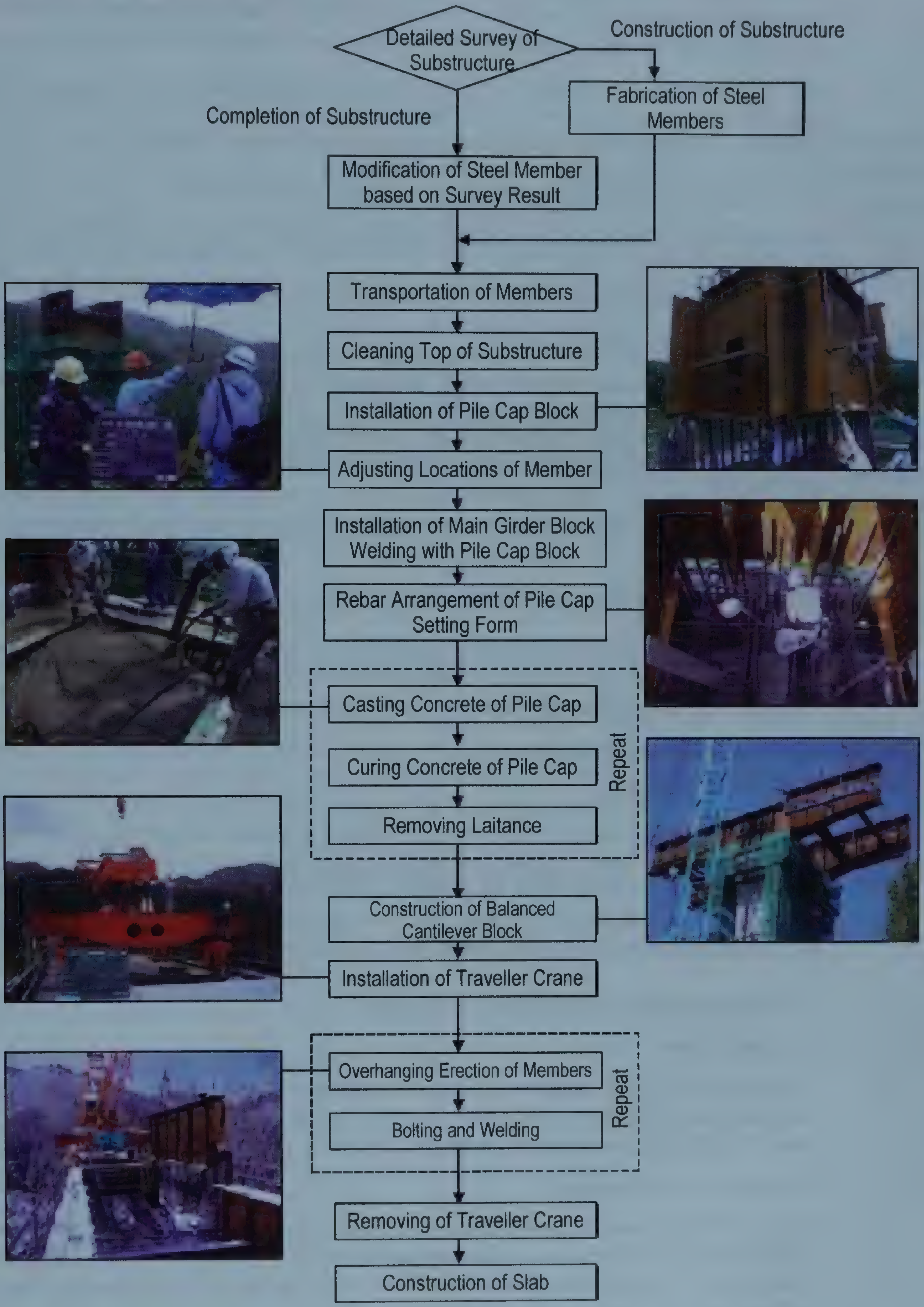

Detailed Survey of Substructure
Construction of Substructure
Fabrication of Steel Members
Completion of Substructure
Modification of Steel Member based on Survey Result
Transportation of Members
Cleaning Top of Substructure
Installation of Pile Cap Block
Adjusting Locations of Member
Installation of Main Girder Block Welding with Pile Cap Block
Rebar Arrangement of Pile Cap Setting Form
Casting Concrete of Pile Cap
Curing Concrete of Pile Cap
Removing Laitance
Repeat
Construction of Balanced Cantilever Block
Installation of Traveller Crane
Overhanging Erection of Members
Bolting and Welding
Repeat
Removing of Traveller Crane
Construction of Slab

Figure 4-22: Construction Procedure of Rigid Connecting Section of Composite Integral (Rigid Framed) Bridge by Overhanging Erection Method with Traveller Crane

4.5.5 Weathering/Weatherproof Steel

(1) Weathering/Weatherproof Steel with stable rust covering the entire exposed surface can be maintenance free member of bridge with specific circumstances which has scarce flying salt amount. Since Himalayan Mountains are far from ocean, there could be regions with ideal environment for weathering/weatherproof steel.

(2) To secure the quality and function of weathering/weatherproof steel, it shall be treated with special care from shipping to construction. This clause describes checking points and notes of handling weathering/weatherproof steel.

(3) Fabrication (cutting, bending, welding, assembling)

Extracted from Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association

Chapter 4 Considerations on Fabrication and Construction

4.2 Considerations on Fabrication

(1) Fabrication and Assembling

Grease and marking shall not be stained on the surface in fabrication in factory. If the steel surface is stained, it shall be cleaned before transportation. Sanding or blasting shall be done by power tools in cleaning grease and marking.

[Source: Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association]

(4) Transportation, Temporary Placement

Extracted from Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association

Chapter 4 Considerations on Fabrication and Construction

4.2 Considerations on Fabrication

(1) Temporary Placement

If the weathering/weatherproof steel member/material is stocked temporarily for a long period, it shall be covered by sheet in order to protect member/material from flying salt adhering to them as much as possible. The location of placement shall be carefully selected considering its minimum height and surrounding condition so as not to wet by rain, drain, mud, or easily fallen down. Long term storage could be a cause of rust and stain, therefore, it shall be blasted (level Sa2) before transportation to site from factory.

4.3 Considerations on Transportation and Erection

(1) Transportation

It is important to check whether salt has been adhered to the surface of weathering/weatherproof steel members during the transportation after receiving them at the site. If the salt is adhered, it shall be washed away with sufficient volume of fresh water and remove the salt on the surface.

(2) Casting Cement Concrete

It is very difficult to remove cement concrete, cement mortar, and sediment completely if those item once has been adhered to the surface of weathering/weather proof steel member. It shall be taken great care for the stain made by material difficult to remove, such as mortar after drying, and wash it away immediately by sufficient amount of fresh water before drying.

It is very difficult to protect the members completely from getting wet by rain at site before erection. However, it could be a cause of uneven corrosion of the surface. Use of surface treatment agent for weathering/weatherproof steel shall be considered depend on the precipitation condition of the site and season.

[Source: Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association]

(5) Surface Treatment

Extracted from Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association

Chapter 5 Construction of Rust Prevention

5.2 Construction

(2) Adjustment of Surface of Steel Members

Mill scale of weathering/weatherproof steel shall be removed before transportation. Mill scale on the surface usually does not have even thickness. It also peels off from the surface unevenly. This could be the uneven progress of corrosion and be difficult to obtain stabile rust. It would be the cause of uneven colour of rust even after the entire mill scale has peeled off. If the mill scale peels off at the site, specifically after erection, it would contaminate the surroundings areas. Considering these facts mentioned above, mill scale of weathering/weatherproof steel shall be removed in fabrication factory before transportation.

There are two method of removing mill scale, one is blasting plate before fabrication and another is blasting product after fabrication. Blasting level is "Surface Preparation Sa2 1/2 (SIS)". Blasting plate is done to remove mill scale of weathering/weatherproof steel plate before fabrication process, such as holing, cutting, welding, in fabrication factory. Blasting with this method can be done easily with high quality but the stain made in fabrication would remain. Blasting product is done except for the section of welding after fabrication when the weathering/weatherproof steel plate has already been fabricated to bridge member. Blasting with this method can transport the product with scares stain with high quality. However, blasting procedure is much more complicate due to existence of welding section. It requires longer period and difficult to control the quality.

[Source: Handbook on Rust Prevention of Road Steel Bridges, Japan Road Association]

(6) Countermeasure by waterproofing and well ventilation

1) Countermeasures to produce even stable rust on the surface of weathering/weatherproof steel members. Securing well drainage, ventilation is the very important as well as protect the members from flying salt. It shall be taken very serious care in the region with snow falls where anti-frozen agent is scattered for road users' safety.

2) Figure below shows the details of Countermeasure by waterproofing and well ventilation.

1) Expansion Joint: Waterproof Type
2) Slab: Covered by Reliable Waterproof Layer
3) Drainage Pipes: Lower the Bottom End, install Flexible Joint
4) Girder End: Cutting Web Plate (Improve Ventilation), Painting
5) Slab Drain (Drain of Slab): Drain lower than girder bottom by hose
6) Drainage Slope on Bottom Flange: Same Angle with Ordinal Steel Girder
7) RCC Riling: Waterproofing at Gap

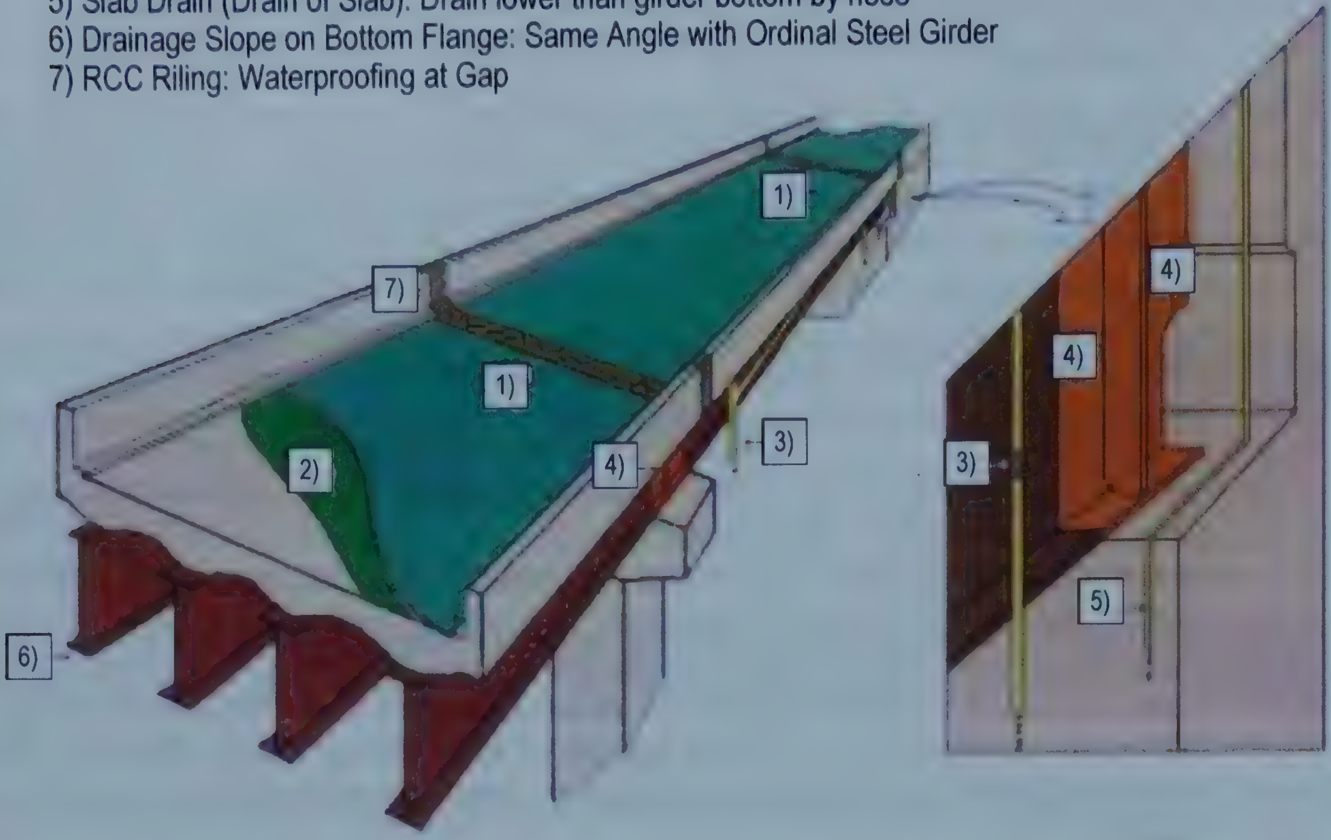


Figure 4-23: Countermeasure by waterproofing and well ventilation

Extracted from Specifications for Highway Bridges of Japan (Part II Steel Bridges)

CHAPTER 18 CONSTRUCTION

18.10 Rust Prevention and Corrosion Prevention

In performing the rust prevention and corrosion prevention work, the items shown from 1) to 5) shall be considered, and the work shall generally be performed so as to secure the prescribed quality.

1) Type and characteristics of rust prevention and corrosion prevention methods
2) Structure and form of work target
3) Work period and work location
4) Work environment condition and the points to be considered
5) Inspection method

Weathering steel is obtained after the quality of the steel itself is improved by adding elements of alloy to the steel, whereupon dense rust generated on the steel surface blocks off water and oxygen, thus restraining the corrosion that takes place afterward. However, the proper repetition of wetting and drying is required for the creation of dense rust, and the usage environment to exert the required performance is limited. Accordingly, in respective stages of work such as delivery and storage as well, care should be taken so that corrosive elements such as salt may not become attached to an excessive degree.

Moreover, when foreign substances, such as mill scale, member marks, and stains that obstruct the formation of dense rusts, exist on the steel surface, the irregularity in the formation of rust arises to spoil the appearance.

[Source: Specifications for Highway Bridges of Japan (Part II Steel Bridges), Japan Road Association]

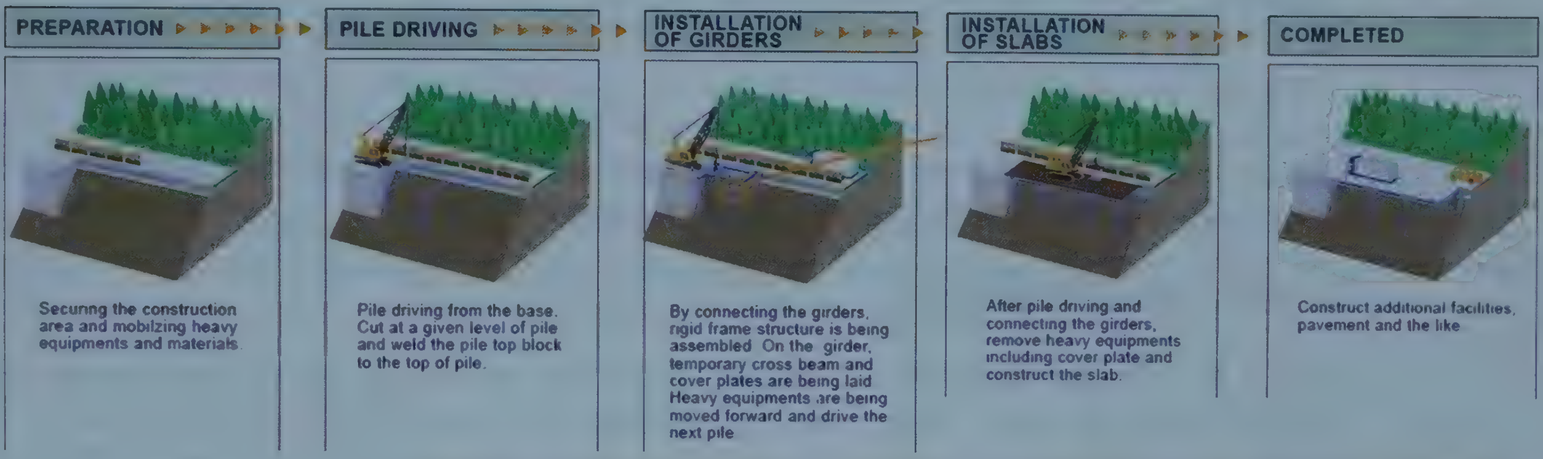
4.5.6 Multipolar Three Dimensional Frame with RC Deck

(1) General

1) Multipolar three dimensional frame with RC deck does not require construction yard at the site. Cranes on the constructed deck does the erection works of steel pile to be inserted to ground as foundation, beams for the RCC deck to be on above, and steel rebars and cement concrete for RCC deck. It gives very little negative impact to the natural environment around the site.

(2) Procedure

1) Construction procedure of multipolar three dimensional frame with RC deck is shown in the figure below.


PREPARATION
PILE DRIVING
INSTALLATION OF GIRDERS
INSTALLATION OF SLABS
COMPLETED
Securing the construction area and mobilzing heavy equipments and materials.
Pile driving from the base. Cut at a given level of pile and weld the pile top block to the top of pile.
By connecting the girders, rigid frame structure is being assembled On the girder, temporary cross beam and cover plates are being laid Heavy equipments are being moved forward and drive the next pile
After pile driving and connecting the girders, remove heavy equipments including cover plate and construct the slab.
Construct additional facilities, pavement and the like

(Source: Handbook of Deep Foundation Construction, Japan Road Association)

Figure 4-24: Construction Procedure of Multipolar Three Dimensional Frame with RC Deck

4.6 River Protection

4.6.1 General

4.6.1.1 River protection for mountain bridges which cross the river is not a miscellaneous member of the bridge. It is one of the most important members to protect the bridge from damage and collapse. The flood of the bridges in India is rapid and massive, which indicate large volume of discharge to derive scour, horizontal force, and carrying debris. Therefore, the construction shall be executed with accuracy and durability. However, this importance is not very different in any terrain.

4.6.1.2 Since the flood discharge is large and the difference with low water is large as well, the protection could be damaged very easily. Therefore, monitoring system or availability of investigation shall be secured after the construction.

4.6.2 Approaches

4.6.2.1 Approaches of bridge can shorten the bridge length and reduce construction cost. However it has high risk of being washed away by flood if it is located in the flood cross section. Therefore, it shall not be constructed solely without other protection structure. It is better to be protected by river training revetment so that the approaches are protected by the revetment and does not face the flood discharge.

4.6.2.2 Making good use of approaches together with river training revetment can reduce both construction cost and period efficiently and effectively. However, responsible bridge engineer together with highway engineer shall keep in mind that approaches are always facing the risk of being washed away. Therefore protection of the approach is essential and also repair and rehabilitation of the approaches shall be prepared during construction because when the approaches are washed away, the river will be close until it is repaired or rehabilitated. It is not enough to protect only the bridge members from food flow.

4.6.3 River Training Works

4.6.3.1 River training works is one of the important structure for river protection. It makes the river flow stable and protects abutments, approaches, and other structures which sustain the bridge or maintain the bridge function. Making the flood discharge stable prevents irregular flow which causes the local scour which could be the reason of the collapse of the bridge.

4.6.3.2 River training works shall not be taken as powerful structure to force flood discharge flow into the flood cross section of the bridge. If the river is trained too strictly, the discharge velocity would be too rapid locally in the flood river section and it would be a cause of severe and unexpected scour or horizontal force. Therefore, the flood cross section shall be secured wide enough to prevent irregular rapid flow so as not to derive unexpected damage to the substructures.

4.6.4 River Protection

4.6.4.1 If the riverbed is fragile and very prone to be washed away, the scouring depth will be very deep and the design of foundation and substructure would be excessive. The river protection together with replacing the riverbed surface soil shall be consider to reduce the construction cost of the bridge.

4.6.4.2 Riverbed protection structure and material shall be selected carefully so as not to be damaged too easily by the flood. Some structure is very fragile that it can be broken by the flood of one monsoon season. Gabion or simple riprap facing rapid and massive flow can easily be broken. Considerable amount of repair and rehabilitation works will be necessary in every dry season. This is waste of maintenance budget and resources and shall be avoided strongly. Therefore selecting the types of river protection works requires the durability factor to the flow velocity which the protection structure is facing. Gabion and simple riprap shall be installed where there mild flow. Solid retaining wall or earth retaining panel with anchors could be the alternative of protecting rapid flow.

4.7 Required Reports

4.7.1 General

4.7.1.1 Recording the construction works is more important for mountain bridges in mountainous and steep terrain than for bridges in other terrain since it have be constructed in more severe weather and fragile surface. The record has to cover the external factors such as climate and water level (both rive and underground) and etc.

4.7.1.2 Since mountain bridges are located in severer natural condition and environment than the bridges in flat terrain, there shall be considerable amount of new findings before commencement of and during the construction, and requires numerous revises/modifications to the design result. All the new findings and revises/modifications shall be recorded clearly so that it could be shared to every relevant personnel of the construction works easily.

4.7.2 Material Test Report

4.7.2.1 Material test procedure and standard shall not be different in mountain bridges with bridges in other terrain. IS and other reliable standard shall be applied to be referred to.

4.7.3 Working Drawings

4.7.3.1 Design drawings shall be revised/modified based on the new findings found before the commencement of the construction of that member or the entire bridge and approved by project owner and other relevant authorities. This shall not be different in mountain bridges with bridges in other terrain.

4.7.4 Construction Working Record

4.7.4.1 Constriction working record shall be recorded in details for every working days of construction. This shall not be different in mountain bridges with bridges in other terrain.

4.7.5 Monitoring report

4.7.5.1 The data of monitoring items at the construction site such as pollution related values shall be recorded and summarized as monitoring report. This shall not be different in mountain bridges with bridges in other terrain.

4.7.6 Monthly Report

4.7.6.1 Construction working record, monitoring record and other necessary record shall be summarized as Monthly Report and submitted to project owner from construction contractor. This shall not be different in mountain bridges with bridges in other terrain.

4.7.7 As-build drawings

4.7.7.1 As-build drawings shall be prepared referring the revises/modifications after the working drawing's approval. Prepared as-build drawings shall be submitted to the project owner by construction contractor. This shall not be different in mountain bridges with bridges in other terrain.

CHAPTER 5 BRIDGE REINFORCEMENT

5.1 General

5.1.1 General

5.1.1.1 Reinforcement is defined as that the existing bridge/member will gain the strength, durability, function more than the original design requirement, or initial condition when the construction is completed. If the original design requirement of initial condition is recovered, it is defined as repair. Repair of the mountain bridges is described in Operation & Maintenance Guideline for Mountain Bridges prepared by JICA Capacity Building Project. Reinforcement might be well-known as 'retrofitting' in some field of construction.

5.1.1.2 Following the definition of the reinforcement, there are not many occasions to apply the reinforcement works. The first occasion is when the design code is upgraded to meet the latest social requirement. There are increase of live load, return period of natural phenomenon such as rainfall, earthquake, wind, and etc. These increase enlarge the design loads of bridges and exceed the limit of the existing bridge to bear the loads.

5.1.1.3 The other occasion is when the existing bridge is found with less strength or/and durability which the existing bridge shall have without being deteriorated. This is due to malconstruction by construction contractor or design error by design consultant. In this case, the strength or durability shall be improved to satisfy the required level from unsatisfied level which does not assure the requirements of the bridge.

5.1.1.4 The strength and/or durability is improved compare to the original status/condition by reinforcement. If it is rehabilitated to the original status/condition, it is only a repair. In this guideline, the reinforcement utilizing CFRP (Carbon Fibre Reinforcement Plastics) and Outer Cable to primary members of the bridge is introduced based on the Japanese examples.

5.1.1.5 Reinforcement against earthquake is another field that Japanese bridge industry has remarkable experience. Apart from increasing the strength of the member, there are Aseismic Bearing/Device and Bridge Fall Prevention Device to improve the earthquake resistance capacity. The former isolates/distributes the seismic force transmitted to substructure from superstructure. The latter control the force or limits the displacement of superstructure by specific devices. In this guideline, both devices are introduced in this chapter based on the Japanese examples.

5.1.2 Conditions

5.1.2.1 To execute the reinforcement to the existing bridge, it is very important mandatory to recognize the design requirements of the bridge in very details. Therefore, the detailed design results, such as main report, calculation sheets, and drawings are necessary to be collected and well-studied.

5.1.2.2 Moreover, it is very important to collect the as-built drawing to comprehend the existing structures condition including invisible interior members/materials, such as re-bars in RCC, tendons/cable in PSC, or inner surface of the steel members with closed cross sections. Trusting that as-built drawing reflects the status of the bridge just after the completion of the construction.

5.1.2.3 The bridge structural members deteriorate gradually by the external environment, such as traffic, rain, wind, flood, earthquake, and chemicals. It specifically deteriorates steel materials, such as re-bars in RCC, tendons/cable in PSC, main and sub-structural members are rusted rapidly and fatally in some cases with snow-melting chemicals in snow-covered regions, such as in high altitude Himalayan Mountains, such as Jammu and Kashmir and Himachal Pradesh. The level of deterioration shall be taken into account for design of the reinforcement. Close investigation with detailed investigation, both at laboratory and site shall be executed to define the exact condition of the existing bridge. Non-destructive inspection shall be executed to comprehend the present status of invisible interior members/materials, such as re-bars in RCC, tendons/cable in PSC, inner surface of the steel members with closed cross sections, or cement concrete.

5.1.2.4 When the members of bridges are heavily deteriorated from the condition of just after the completion of the construction, in other words, lack the requirements in strength, durability, function, and safety, it shall be repaired before the commencement of the reinforcement work to be executed. Failure of this repair would not secure the improved strength, durability, function, and safety after the completion of the reinforcement works. To execute the repair, the present condition shall be recognized in details and repair works shall be designed and constructed properly to secure the required strength, durability, function, and safety which the bridge should have in reinforcement works.

5.1.2.5 There shall be many old bridges without detailed design results nor as-built drawings. The present condition shall be defined by studying the inspection results. However, this definition is based on the assumption derived from the inspection and application to members with similar condition due to the limitation of the inspection budget. Therefore, the design result for the reinforcement requires frequent review by applying the new findings which are found during the actual construction work of reinforcement.

5.1.2.6 Defining the present condition by many steps and also referring to the design until the very last moment of reinforcement works are essential to gain the benefit of the reinforcement works. This is not unique to the mountain bridges but also to the bridges in any other terrains. However, the mountain bridges suffer the severe external conditions which deteriorate the bridge member rapidly and severely. Therefore, the definition of the present condition of the exiting mountain bridges shall be executed with careful attention and review, and revising shall be done as many times as possible.

5.1.3 Scope

5.1.3.1 There are three main structural members of bridges, such as superstructure, substructure, and foundation. There are miscellaneous non-structural members to secure the safety and comfortability to the passing traffic and surrounding area. The reinforcement could be executed to any of the members of the bridge. However, it is realistic to replace the miscellaneous non-structural members instead of executing reinforcement works. Therefore, this guideline treats the main structural members and does not describe the reinforcement of the miscellaneous members.

5.1.3.2 Reinforcement of the foundation are divided into two major methods. One is to strengthen the foundation by enlarging size or increasing numbers, such as enlarging the footing of spread foundation or increasing the numbers of piles. The other is to improve the subsoil condition by mixing hardening material or replacing. The former method could be executed to mountain bridges and is based on the very classical method of integrating the exiting and additional footing together and the new technology is focused on the connecting material or equipment, such as adhesive, joint coupler, admixtures, and etc. The latter is likely to be applied to the soft subsoil in flat terrain. Therefore, reinforcement of foundation is excluded from the contents of this guideline.

5.1.3.3 This guideline treats the reinforcement of superstructure and substructure which is made by RCC, PSC and steel members in general. It also explains the reinforcement of composite members. It also focuses on the contents which special attention is required for mountain bridges.

5.1.4 Objectives

5.1.4.1 Objectives of reinforcement are either or multiple of the followings,

(1) To strengthen the member(s) of the bridge and the entire bridge system

(2) To improve the durability of the member(s) of the bridge and the entire bridge system

(3) To improve the safety of the entire bridge system

5.1.4.2 Objectives of this chapter is to introduce the state-of-art technologies and example of its application of reinforcement of superstructure and substructure.

5.1.4.3 The other objective is to clarify the needs and countermeasure of the reinforcement of the mountain bridges in India.

5.1.5 Requirements

5.1.5.1 Requirements to commence the reinforcement project of the existing bridges are as follows,

(1) Collecting the original documents and drawings (detailed design and construction stage) and understanding them to recognize the original status and conditions. Required documents and drawings are detailed design report, structural calculation sheets, and as-built drawings, and etc.

(2) Target level of improvement, such as revised design standard, new technical specification, and etc.

5.1.5.2 Requirements to commence the detailed design of the reinforcement of the existing bridges are as follows,

(1) Execution of close inspection at laboratory and site, non-destructive inspection shall be executed to define the level of deterioration with its report in details.

(2) Results of necessary laboratory test to define the deterioration of members of the existing bridge.

(3) Site reconnaissance

(4) Detailed definition of present condition of the existing bridge based on the collected documents and drawings and result of inspection and tests.

5.1.5.3 Requirements to commence the construction works of reinforcement of the existing bridges are as follows,

(1) Results of detailed design of reinforcement of the existing bridges.

(2) Organization structure with appropriate engineers/specialists to execute the review and revision to the results of detailed design with new findings found during the reinforcement works.

(3) Budget allowance for amendment for project owner.

5.1.6 Procedure

5.1.6.1 The following figure shows the procedure of the reinforcement of bridge.

5.1.6.2 Responsible bridge engineer shall keep in mind that in reinforcement project, it is very difficult to collect and comprehend the present status and condition of members of the existing bridge and there would always be new findings which necessitate the review of the result of detailed design. Members of mountain bridges are usually difficult to access and the data collected before the commencement of the construction is very limited due to the budget limitation. Therefore, construction constructor shall be ready for revision of detailed design during the reinforcement works to secure the requirement of the reinforcement project.

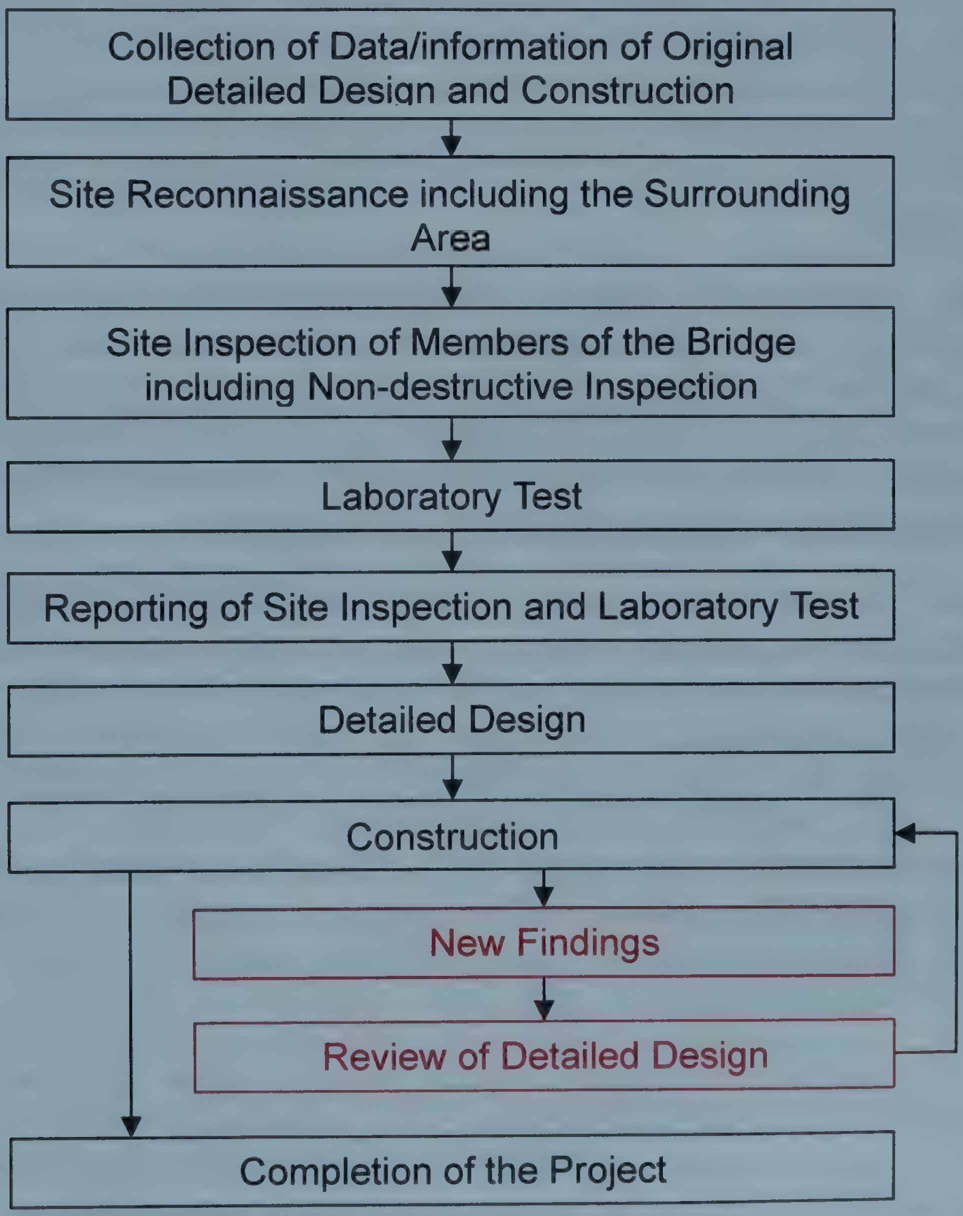

Collection of Data/information of Original Detailed Design and Construction
Site Reconnaissance including the Surrounding Area
Site Inspection of Members of the Bridge including Non-destructive Inspection
Laboratory Test
Reporting of Site Inspection and Laboratory Test
Detailed Design
Construction
New Findings
Review of Detailed Design
Completion of the Project

Figure 5-1: Procedure of the Reinforcement Project of Bridge

5.2 Superstructure

5.2.1 General

5.2.1.1 Reinforcement of the superstructure depends on the structural status of superstructure of the bridge. If the superstructure is sustained by the bearing on the top of the substructure, the most of the case in reinforcement of superstructure is improving the performance against the live load, such as strengthening the capacity of superstructure and increasing the bearing force against bending moment and shear.

5.2.1.2 If the superstructure is rigidly connected to substructure, the reinforcement would not be as simple as the case of the superstructure sustained by bearings on the top of substructures. There would be other combinations of load which are necessary to be considered to design the reinforcement of the members. It is often necessary that it shall be reinforced together with substructure. The scale of the project would be relatively large.

5.2.1.3 If the bridge has the rigid structure of superstructure and substructure connected together and the number/area of reinforcement section/member becomes large, the increase amount of the dead load would be above the limit to maintain the stability of foundation. Responsible bridge engineer shall review the stability of the foundation in selecting the reinforcement method, specifically in the bridges with rigid connection of superstructure and substructure. Mountain bridges with high design speed of traffic, rigid connection of superstructure and substructure is often applied with long span length.

5.2.1.4 Reinforcing major mountain bridges with long span length and rigid connection of superstructure and substructure, requires large size of equipment and temporary structure (scaffolding, support, jetty, and etc.) for reinforcement works. Not only the reinforcement method to be applied but also the construction planning, including the equipment and temporary structure shall be studied carefully in detailed design stage for the reinforcement project of mountain bridges.

5.2.1.5 Superstructure sustain the traffic directly. Therefore, it is damaged by the impact derived by the heavy vehicles which passes the bridge with high speed specifically when there are gaps on the surface of the bridge above the pavement. The gap could be vertically, horizontally, or 3-dimentionally in both direction combined, that can damage the superstructure severely. If there is other driver of deterioration such as chemicals, malconstruction, insufficient materials, the deterioration could be progressive much more rapidly than without those deterioration drivers. Therefore, it is very important to recognize the present condition of the superstructure before and during the execution of reinforcement works to secure the requirement of the reinforcement works.

5.2.1.6 Mountain bridges are often very far from maintenance centre of the highway operation organization. Therefore, the damage of the surface and member itself are prone to be unfound or ignored. This could accelerate the deterioration very easily. Therefore, it is important to understand the maintenance condition of the bridge to recognize and define the present condition of the bridge which are to be reinforced.

5.2.2 Methods

5.2.2.1 There are basically five methods to reinforce superstructure excluding replacement of members or superstructure itself. Five methods are as follows,

(1) Steel Plate Bonding Method

(2) CFRP Bonding Method

(3) Cross-section Enlargement Method

(4) Additional Outer Cable Method

(5) Additional Girder/Supporting Point Method

5.2.2.2 Method (1), (3), and (5) are based on the very classical method to reinforce strength, durability, function and safety of superstructure. The new technology is focused on the connecting material or equipment, such as adhesive, joint coupler, admixtures, and etc. Introduction of these technologies are dependent on the manufactures of the materials or equipment. Since this guideline treats general aspects, these method is excluded from this guideline's contents.

5.2.2.3 This guideline treats method (2) and (4) which are relatively new methods for reinforcing the superstructure, focusing the example of the reinforcement in Japan. Japan suffer the damage and deterioration of RCC deck for decades due to the thin thickness of the RCC deck. However, bridges in India rarely have same damages of RCC deck with bridges in Japan because it has enough thickness of RCC deck to sustain military tanks. Therefore, the reinforcement method of RCC deck is excluded from this guideline's contents.

5.2.3 Outer Cable

5.2.3.1 Outline

(1) Reinforcement of superstructure by outer cable is defined that the outer cable attached to the existing superstructure provides prestressed compressive force transmitted through anchorage zone and deflection device.

(2) It improves the bearing capacity against bending moment.

(3) It could be applied to RCC, PSC, and Steel superstructure.

(4) It could be applied to various shapes of cross section, such as box-girder, I-girder, T-girder, voided slab, filled slab.

(5) Outer cables are given tension force by prestressing and provide compressive force to the exiting superstructure, it reduces the tension stress in tension member/section.

(6) It is able to provide prestress force with small increase of dead load because the outer cable with its attachment equipment is relatively small.

(7) The existing superstructure shall have structural soundness to bear the prestress force, compressive force. The present condition of the existing superstructure shall be verified by the detailed inspection. If the superstructure is deteriorated and are not able to bear the prestress compressive force, the soundness of the deteriorated member shall be recovered before the reinforcement work commence.

(8) It shall be checked at the site inspection that there are sufficient space to attach outer cable including anchorage/deflection device.

(9) Outer cable shall be well coated to prevent corrosion. A typical and well-applied method of corrosion prevention is coat with anti-corrosion oil and covered by polyethylene resin.

(10) Anchorage zone shall be reinforced by additional rebar to bear the concentration force of prestress. Sufficient space for prestressing shall be secured behind the anchorage zones.

(11) Re-prestressing shall be able after the reinforcement project for maintenance.

5.2.3.2 Procedure

(1) The procedure of the reinforcement of superstructure by outer cable has same procedure with Figure 5.1 in defining the present condition of the superstructure, such as from collection data/information of original detailed design and construction to reporting of site reconnaissance and laboratory test. The procedure of detailed design and construction is as follows,

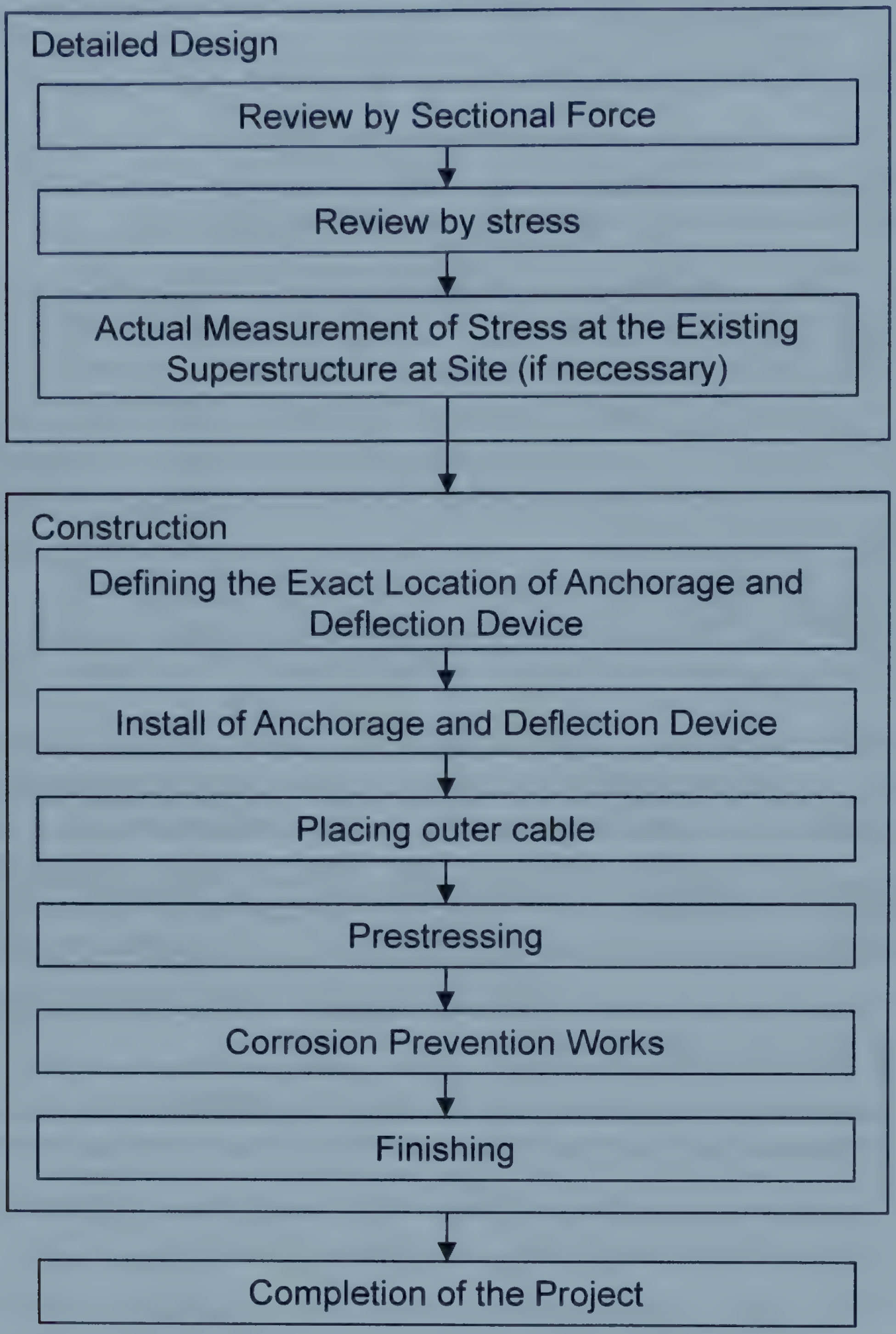

Detailed Design
Review by Sectional Force
Review by stress
Actual Measurement of Stress at the Existing Superstructure at Site (if necessary)
Construction
Defining the Exact Location of Anchorage and Deflection Device
Install of Anchorage and Deflection Device
Placing outer cable
Prestressing
Corrosion Prevention Works
Finishing
Completion of the Project

Figure 5-2: Procedure of the Reinforcement of Superstructure by Outer Cable

5.2.3.3 Example

5.2.4 CFRP

5.2.4.1 Outline

(1) There are two methods of reinforcement of superstructure by CFRP. One is bonding CFPR sheets to the surface of the superstructure, and the other is CFRP plate attached to existing structure with prestress of tension force. The latter has same methodology with reinforcement by outer cable which is mentioned in the previous clause.

(2) Reinforcement by CFRP sheet improves the bearing capacity against bending moment and/or sheer. It depends on the location and surface where the CFRP are bonded.

(3) Reinforcement by CFRP plate improves the bearing capacity against bending moment. It is basically attached to the bottom surface of the girder/slab with much less space compared to outer cable.

(4) It could be applied to RCC, PSC, and Steel superstructure.

(5) It could be applied to various shapes of cross section, such as box-girder, I-girder, T-girder, voided slab, filled slab.

(6) In case the CFRP plate are prestressed and tension force is given, it provides compressive force to the exiting superstructure, it reduces the tension stress in tension member/section.

(7) It is able to provide prestress force with very scarce increase of dead load because the CFRP plate with its attachment equipment is very small.

(8) The existing superstructure shall have structural soundness to bear the prestress force, compressive force. The present condition of the existing superstructure shall be verified by the detailed inspection. If the superstructure is deteriorated and are not able to bear the prestress compressive force, the soundness of the deteriorated member shall be recovered before the reinforcement work commence.

(9) It shall be checked at the site inspection that there are sufficient space to attach outer cable including anchorage/deflection device.

(10) Anchorage zone shall be reinforced by additional rebar to bear the concentration force of prestress. Sufficient space for prestressing shall be secured behind the anchorage zones.

(11) Re-prestressing shall be able after the reinforcement project for maintenance.

(12) In case the CFRP plate/sheet is bonded to the surface of the existing superstructure, the uneven surface shall be well corrected to transmit the force to the CFRP plate/sheet.

(13) CFRP plate/sheet shall be covered by protection coating. Coating material is whether resin or mortar.

5.2.4.2 Procedure

(1) The procedure of the reinforcement of superstructure by CFRP with prestressing has same procedure with Figure 5.1 in defining the present condition of the superstructure, such as from collection data/information of original detailed design and construction to reporting of site reconnaissance and laboratory test. The procedure of detailed design and construction is as follows,

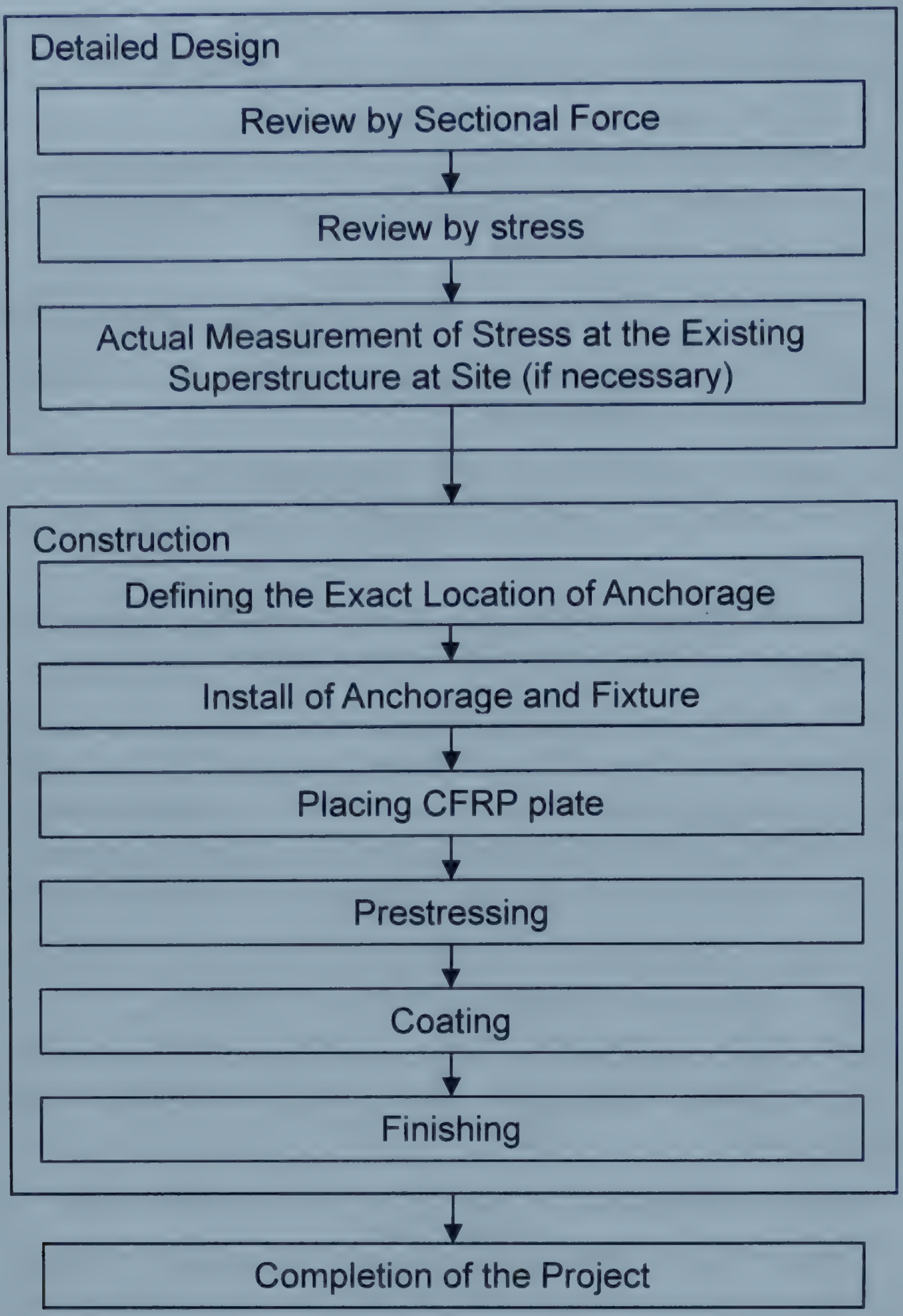

Detailed Design
Review by Sectional Force
Review by stress
Actual Measurement of Stress at the Existing Superstructure at Site (if necessary)
Construction
Defining the Exact Location of Anchorage
Install of Anchorage and Fixture
Placing CFRP plate
Prestressing
Coating
Finishing
Completion of the Project

Figure 5-3: Procedure of the Reinforcement of Superstructure by CFRP with Prestressing

(2) The procedure of the reinforcement of superstructure by CFRP bonding also has same procedure with Figure 5.1 in defining the present condition of the superstructure, such as from collection data/information of original detailed design and construction to reporting of site reconnaissance and laboratory test. The procedure of detailed design and construction is as follows,

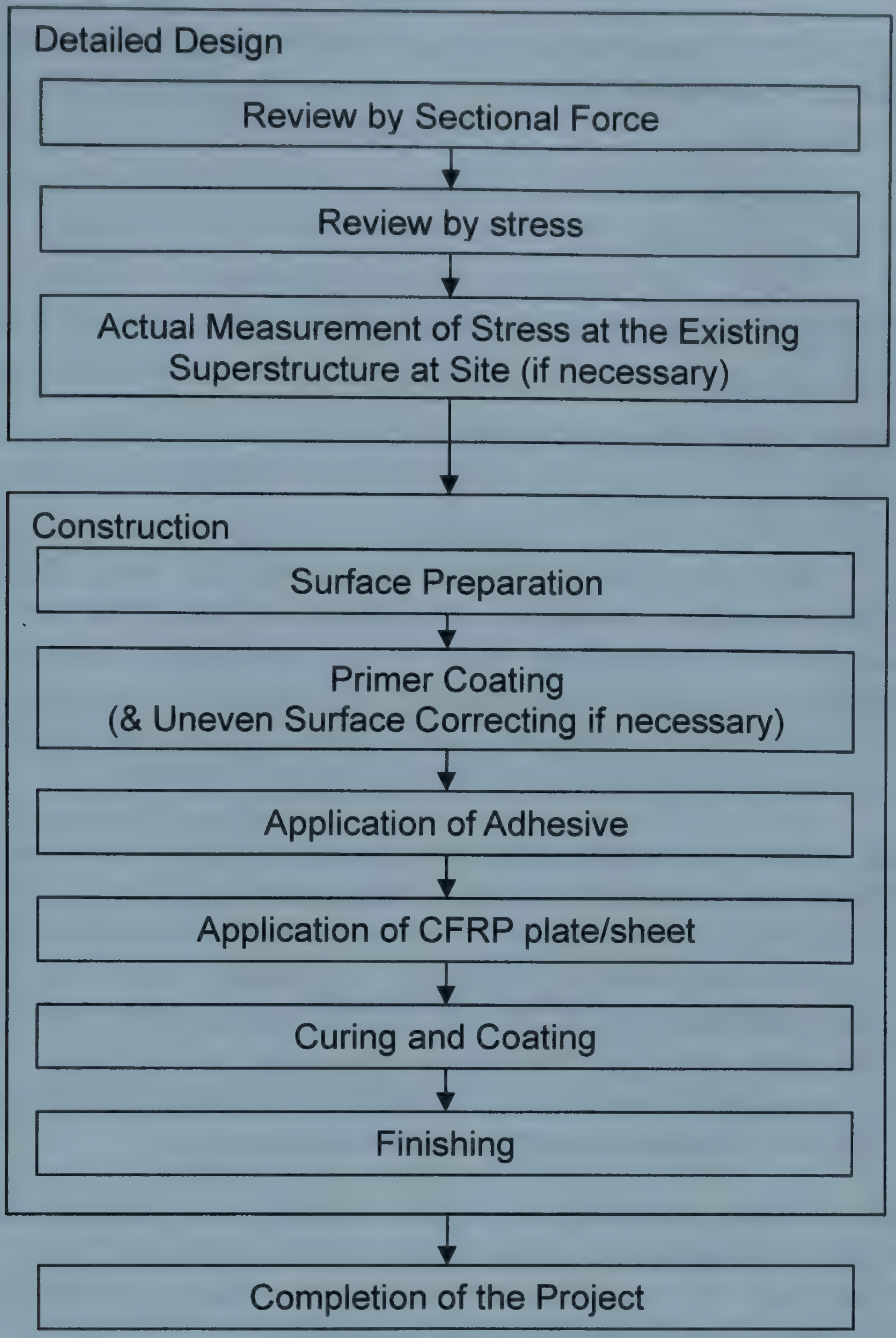

Detailed Design
Review by Sectional Force
Review by stress
Actual Measurement of Stress at the Existing Superstructure at Site (if necessary)
Construction
Surface Preparation
Primer Coating (& Uneven Surface Correcting if necessary)
Application of Adhesive
Application of CFRP plate/sheet
Curing and Coating
Finishing
Completion of the Project

Figure 5-4: Procedure of the Reinforcement of Superstructure by CFRP Bonding

5.2.4.3 Example

5.3 Substructure

5.3.1 General

5.3.1.1 Reinforcement of the substructure is required basically by the upgrade of the design code and require larger bearing capacity of load transmitted from superstructure. The load could be derived by earthquake, wind, and flood discharge. The upgrade could be done because of the needs of the society increased and request safer bridges. This could be reflected to the extension of the return period. The other reason of upgrade could be the historical record breaking natural phenomenon of earthquake, wind, rainfall. This phenomenon revises the design load of the bridge defined in the design code.

5.3.1.2 No matter what is the upgrade of the code is derived from, the reinforcement of the substructure is improving the capacity to resist against bending moment, torsion moment or/and shear. It could be realized by improving the capacity of the substructure or reducing the transmittal of load from superstructure. The methods of reinforcing substructure and reducing the load transmitted from superstructure are mentioned in the clause below.

5.3.2 Methods

5.3.2.1 There are basically tree methods to reinforce substructure excluding replacement of members or superstructure itself. Three methods are as follows,

(1) RCC Rapping Method

(2) Steel Plate Rapping Method

(3) CFRP Rapping Method

5.3.2.2 Method (1) and (2) are based on the very classical method to reinforce strength, durability, function and safety of superstructure. The new technology is focused on the connecting material or equipment, such as adhesive, joint coupler, admixtures, and etc. Introduction of these technologies are dependent on the manufactures of the materials or equipment. Since this guideline treats general aspects, these method is excluded from this guideline's contents.

5.3.2.3 This guideline treats method (3) which are relatively new methods for reinforcing the superstructure, focusing the example of the reinforcement in Japan. Most of the example of Japan is reinforcement of bearing capacity against earthquake, since Japan has suffered massive earthquake since dawn of history. There were tremendous damage to the people and society by Great Hanshin Earthquake in 1995 and the Great East Japan Earthquake in 2011. The bridge design code was upgraded several times to cope with the issue to secure safety of the bridge against massive load derived from the new historical record earthquake.

5.3.2.4 There are basically two methods to reduce the load transmitted from superstructure excluding replacing the entire or part of the superstructure and reducing the dead load. It is applied to the superstructure sustained by bearings on the top of substructures. Two methods are as follows,

(1) Aseismic Bearing/Device Method

(2) Bridge Fall Prevention Method

5.3.2.5 There are three types of aseismic bearing/device, such as Seismic Isolation bearing, damper, and load distributing bearing. This method is introduced in this guideline focusing the examples in Japan which are installed additionally to the existing bridges recently.

5.3.2.6 Bridge fall prevention method prevent superstructure to fall down when the massive earthquake disaster. There are three approaches to install the bridge fall prevention structure/device, such as enlarging the area of top surface of the substructure, installing fall prevention devices, and installing displacement control structure. This method is introduced in this guideline focusing the examples in Japan which are installed additionally to the existing bridges recently.

5.3.3 CFRP

5.3.3.1 Outline

(1) Unlike the reinforcement of superstructure, reinforcement of substructure by CFRP basically is not with prestressing. Therefore, it is rare that CFRP plate is applied. CFRP sheet is bonded to surface of substructure, such as column or wall.

(2) When the substructure has rigid frame structure, and the beam section required reinforcement against bending moment, the reinforcement by CFRP with prestress would be an alternative method to be applied.

(3) Reinforcement by CFRP sheet improves the bearing capacity against bending moment and/or sheer. It depends on the location and surface where the CFRP are bonded.

(4) It could be applied to RCC, PSC, and Steel substructure. However, it is applied to RCC in case of reinforcement of substructure.

(5) The surface of the existing superstructure which CFRP sheet is bonded, the uneven surface shall be well corrected to transmit the force to the CFRP sheet.

(6) CFRP sheet shall be covered by protection coating. Coating material is whether resin or mortar.

5.3.3.2 Procedure

(1) The procedure of the reinforcement of substructure by CFRP bonding has same procedure with Figure 5-4. The procedure is as follows,

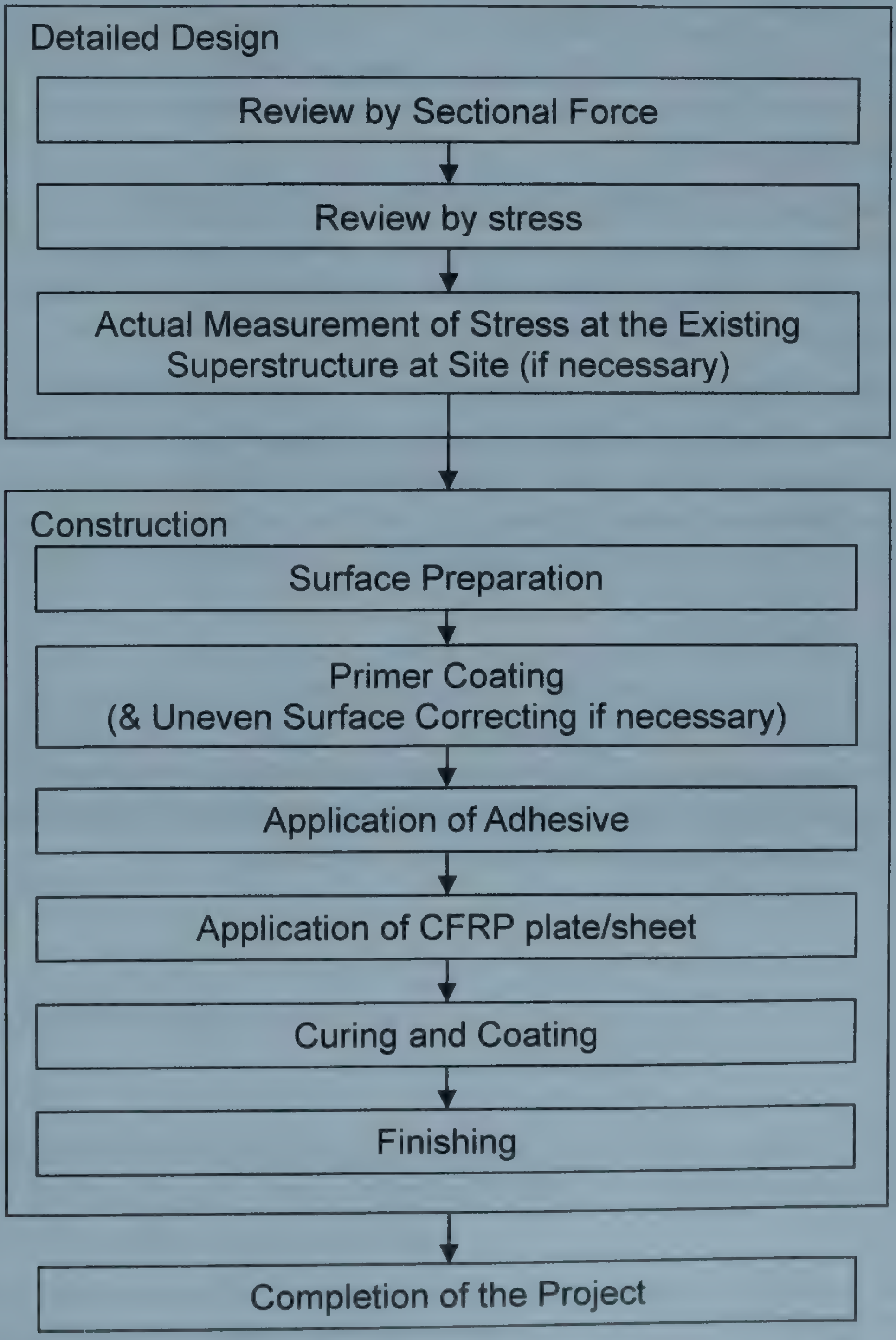

Detailed Design
Review by Sectional Force
Review by stress
Actual Measurement of Stress at the Existing Superstructure at Site (if necessary)
Construction
Surface Preparation
Primer Coating (& Uneven Surface Correcting if necessary)
Application of Adhesive
Application of CFRP plate/sheet
Curing and Coating
Finishing
Completion of the Project

Figure 5-5: Procedure of the Reinforcement of Substructure by CFRP Bonding

5.3.3.3 Example

5.3.4 Aseismic Bearing/Device

5.3.4.1 Outline

(1) Bridges could be reinforced by installing higher grade aseismic bearing/device. It reduces the load transmitted from superstructure to substructure and improves not only the bearing capacity of substructure but also the stability of the entire bridge.

(2) There are basically three types of grade aseismic bearing/device as follows,

1) Seismic Isolation Bearing

2) Damper

3) Load Distributing Bearing

(3) Seismic Isolation Bearing reduces the load transmitted to substructure from superstructure by installing Lead Rubber Bearing (LRB). LRB prolongs the period of earthquake vibration by laminated rubber bearing. It also damps the earthquake vibration by lead plug which is set internal of the laminated rubber bearings. The ductility of lead plug damps the earthquake vibration rapidly. The restorability of rubber bearing return the superstructure to the location where it was placed before the earthquake occurred.

(4) Dampers are installed connected both to superstructure and substructure to damp the relative displacement/movement of the superstructure to substructure/ground. There are basically three mechanism of damping. 1st is damping by frictional resistance, 2nd is hydraulic pressure, and the 3rd is plastic deformation.

(5) Load Distributing Bearing are laminated rubber bearings which replaces fixed/move bearings of the existing bridge. It distributes the load transmitted from superstructures which was transmitted to single substructure with fixed bearing to multiple substructures. Reinforcing of the substructures would be much more simple than that of the case without replacing the fixed/move bearings.

5.3.4.2 Procedure

(1) The procedure of the reinforcement of substructure by upgrading the existing bearings also has same procedure with Figure 5.1 in defining the present condition of the superstructure, such as from collection data/information of original detailed design and construction to reporting of site reconnaissance and laboratory test. The procedure of detailed design and construction is as follows,

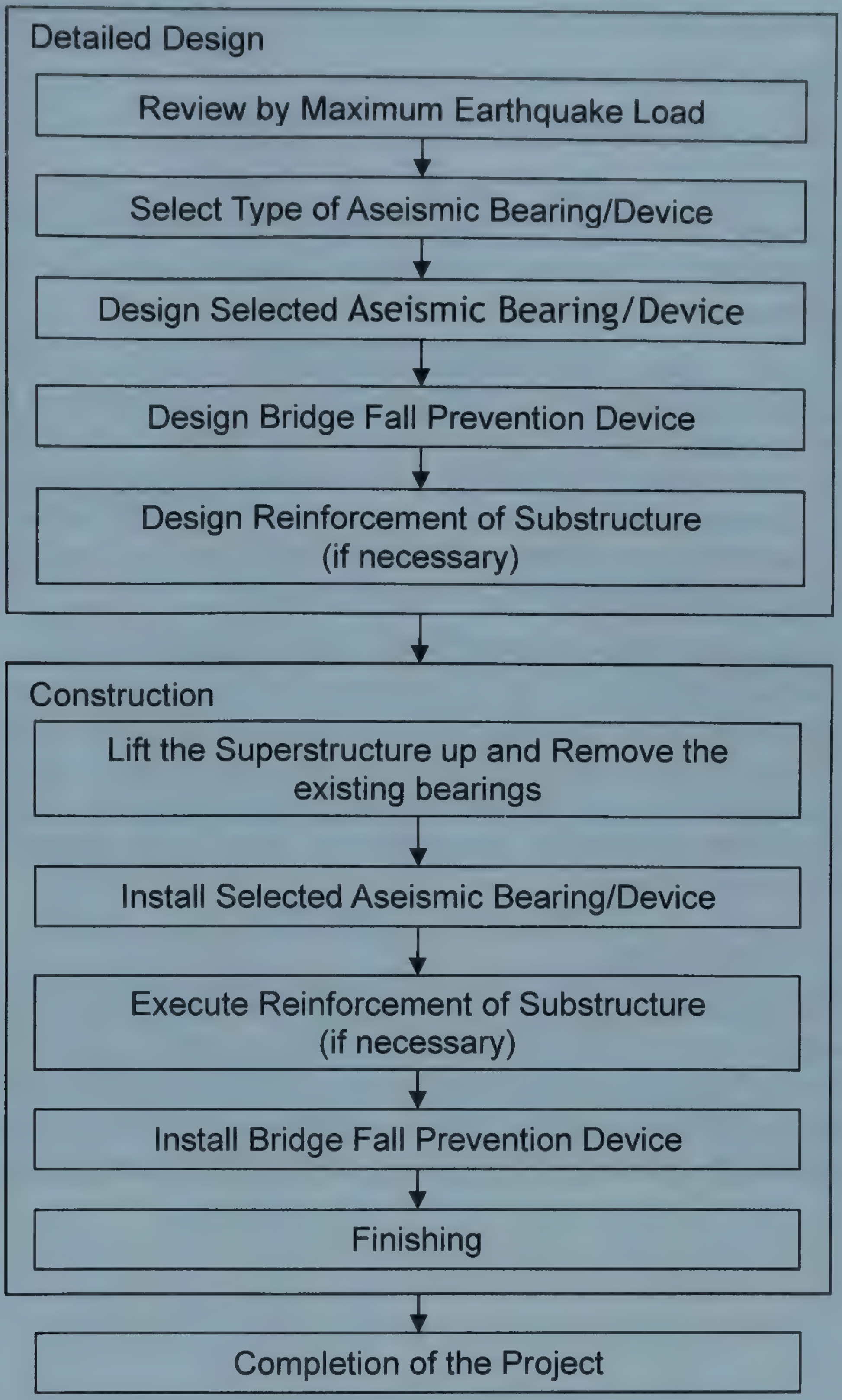

Detailed Design
Review by Maximum Earthquake Load
Select Type of Aseismic Bearing/Device
Design Selected Aseismic Bearing/Device
Design Bridge Fall Prevention Device
Design Reinforcement of Substructure (if necessary)
Construction
Lift the Superstructure up and Remove the existing bearings
Install Selected Aseismic Bearing/Device
Execute Reinforcement of Substructure (if necessary)
Install Bridge Fall Prevention Device
Finishing
Completion of the Project

Figure 5-6: Procedure of the Reinforcement of Substructure by Replacing the Existing Bearings with Aseismic Bearing/Device

5.3.4.3 Example

5.3.5 Bridge Fall Prevention Device

5.3.5.1 Outline

(1) Bridges could be reinforced by bridge fall prevention device. It limits the relative displacement/movement of the superstructure to substructure and prevents the superstructure from collapsing to the ground.

(2) It is often applied to the emergency arterial routes in natural disaster since it is able to open the route rapidly after the disaster because it limits the gaps between superstructures or superstructures and substructures in a limited scale.

(3) There are basically three types/methods of grade bridge fall prevention device as follows,

1) Securing Seating Length of Superstructure on Substructure

2) Bridge Fall Prevention Structure

3) Transversal Displacement Restraint Structure

(4) Sufficient seating length is defined that the maximum relative displacement/movement of superstructure to substructure in case of maximum design earthquake load. It prevents the superstructure from collapsing from substructure after the bearings are destroyed.

(5) Bridge fall prevention structure limits the longitudinal relative displacement/movement of superstructure to substructure to seating length after the bearings are destroyed in case of maximum design earthquake load. There are basically two methodologies of restricting the relative displacement/movement of superstructure to substructure. One is by tension resistance and the other is compression resistance. The former has its material as PSCable, steel rod, or steel chain. The latter's material are rubber (shock absorber) and steel (stopper).

(6) Transversal displacement restraint structure limits the transversal relative displacement/movement of superstructure to substructure to calculated value in detailed design after the bearings are destroyed in case of maximum design earthquake load. It shall be applied to the bridges with keen skew angle, small radius, or substructure with very narrow superstructure seat/pier head. The basic methodology, structure, and material is basically the same with compression resistant bridge fall prevention structure.

5.3.5.2 Procedure

(1) The procedure of the reinforcement of bridge function by installing bridge fall prevention device also has same procedure with Figure 5.1 in defining the present condition of the superstructure, such as from collection data/information of original detailed design and construction to reporting of site reconnaissance and laboratory test. The procedure of detailed design and construction is as follows,

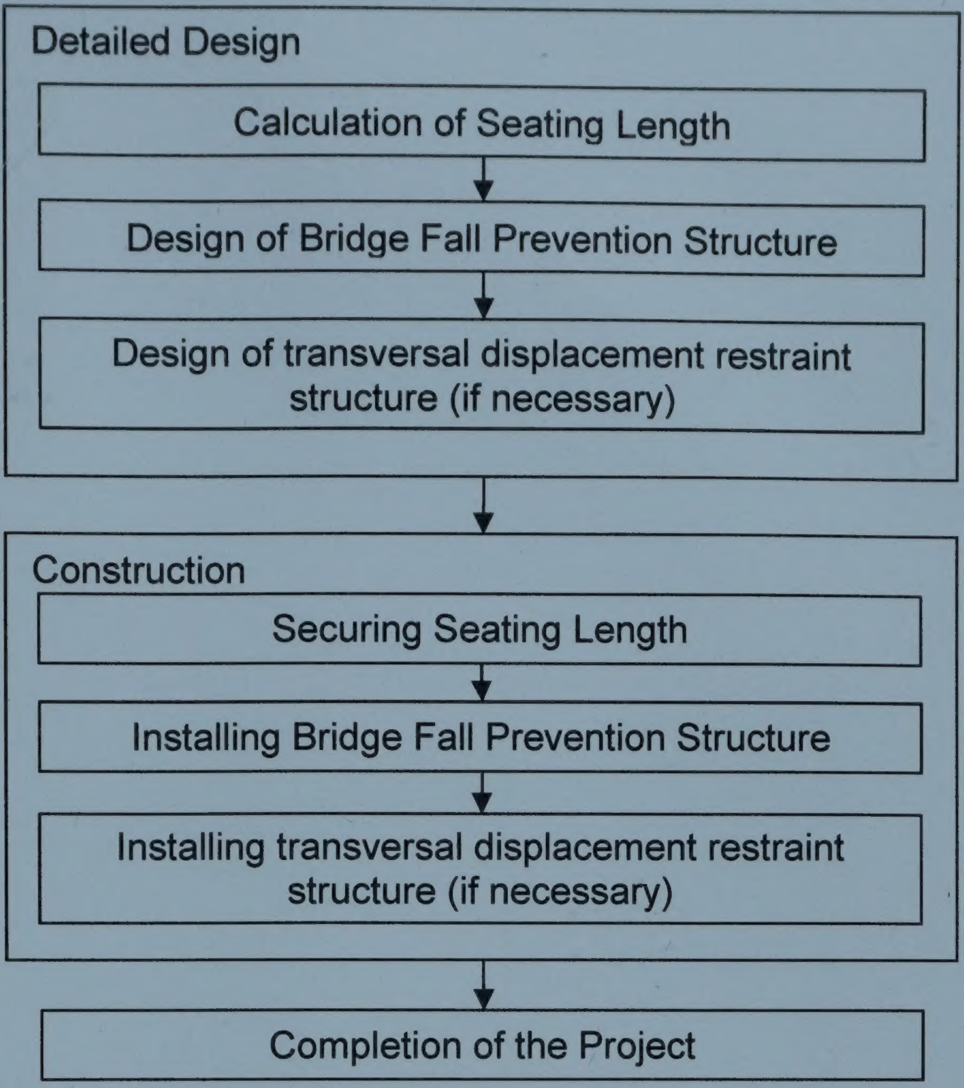

Detailed Design
Calculation of Seating Length
Design of Bridge Fall Prevention Structure
Design of transversal displacement restraint structure (if necessary)
Construction
Securing Seating Length
Installing Bridge Fall Prevention Structure
Installing transversal displacement restraint structure (if necessary)
Completion of the Project

Figure 5-7: Procedure of the Reinforcement of Bridge Function by Installing Bridge Fall Prevention Device